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SCREENING/FLOTATION TREATMENT OF COMBINED SEWER OVERFLOWS

Donald G. Mason, et al

Rex Chainbelt, Incorporated

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This report documents a study to develop a treatment system for combined sewer overflows. The processes of chemical oxidation, screening, dissolved-air flotation, and disinfection were evaluated in the laboratory. A 5 MGD demonstration system was designed, installed and evaluated.

The system was utilized to treat 55 combined sewer overflows. The drainage area served by the system was a 500 acre completely developed residential area of Milwaukee, Wisconsin. Suspended solids and volatile suspended solids removal in the range of 65-80% were consistently obtained at influent concentrations of 150 to 600 mg/l. BOD and COD removals were slightly lower at 55 to 65% for influent concentrations of 50 to 500 mg/l. Addition of chemical flocculents (ferric chloride and a cationic polyelectrolyte) was necessary to obtain these removals. Without the use of chemical flocculents, removal of BOD, COD, suspended solids, and volatile suspended solids were all in the range of 40-50%. The screening flotation system provided sufficient detention time (~15 minutes) for adequate disinfection with hypochlorite salts. Cost estimates indicate a capital cost of \$21,056 per MGD capacity or \$3,828 per acre for a 90 MGD screening/flotation system. Operating costs were estimated at 3.09¢/1000 gallons of treated overflow, including chemical flocculent addition.
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SCREENING/FLOTATION TREATMENT

OF

COMBINED SEWER OVERFLOWS

by

The Ecology Division
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Milwaukee, Wisconsin

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ABSTRACT

This report documents a study to develop a treatment system for combined sewer overflows. The processes of chemical oxidation, screening, dissolved-air flotation and disinfection were evaluated in the laboratory. It was determined that chemical oxidation was not feasible. The majority of the pollutants were of a particulate nature, which indicated solids/liquid separation processes could possibly provide effective treatment. A 5 MGD combination screening and dissolved-air flotation demonstration system was designed, installed and evaluated.

The system was utilized to treat 55 combined sewer overflows. The drainage area served by the facility was a completely developed 500 acre residential area of Milwaukee, Wisconsin. Suspended solids and volatile suspended solids removal in the range of 65-80% were consistently obtained at influent concentrations of 150 to 600 mg/l. BOD and COD removals were slightly lower at 55 to 65% for influent concentrations of 50 to 500 mg/l. Addition of chemical flocculents (ferric chloride and a cationic polyelectrolyte) was necessary to obtain these removals. Without the use of chemical flocculents, removal of BOD, COD, suspended solids, and volatile suspended solids were all in the range of 40-50%. The screening/flotation system provided sufficient detention time (~15 minutes) for adequate disinfection with hypochlorite salts. Cost estimates indicate a capital cost of $21,056 per MGD capacity or $3,828 per acre for a 90 MGD screening/flotation system. Operating costs were estimated at $3.09¢/1000 gallons of treated overflow, including chemical flocculent addition.

This report was submitted as partial fulfillment of contract number 14-12-40 under the sponsorship of the Water Quality Office Environmental Protection Agency.

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SECTION I

CONCLUSIONS

The following conclusions can be made based on the data obtained during this study.

1. A screening/flotation treatment system is an effective method of reducing the pollution caused by combined sewer overflows.

2. The combined sewer overflows monitored during this study had the following characteristics in mg/l at the 95% confidence level.

BOD	49±10
COD	161±19
Suspended Solids	166±26
Volatile Suspended Solids	90±14
Total Nitrogen	5.5±0.8

3. Approximately 20% of the overflows monitored exhibited first flushes of the following characteristics in mg/l at the 95% confidence level.

BOD	186±40
COD	581±92
Suspended Solids	522±150
Volatile Suspended Solids	308±83
Total Nitrogen	17.6±3.1

When present these first flushes persisted for 20 to 70 minutes.

4. All first flushes occurred at a time interval of greater than 4 days between overflows.

5. It was demonstrated that a screening/flotation system could achieve the following percent removal efficiencies based on the raw waste characteristics encountered(Hydraulic Loading:Screen 40 gpm/sq ft,Flotation 2.75 gpm/sq ft).

	Without Chemical Flocculant Addition	With Chemical Flocculant Addition
BOD	35±8	60±11
COD	41±8	57±11
Suspended Solids	43±7	71±9
Volatile Suspended Solids	48±11	71±9
Nitrogen	29±14	24±9

6. The chemical flocculant addition required to achieve the above stated removals was 20 mg/l ferric chloride and 4 mg/l of a cationic polyelectrolyte.

7. An operating pressure of 50 psig and a pressurized flow rate of 20% of the raw waste flow provided sufficient air bubbles for proper flotation tank operation.

8. The average volume of waste residual solids (i.e. screen backwash and floated scum) was 1.75% of the raw flow.

9. The average removal efficiencies for the screening/flotation system decreased (as shown below in percent removals) when the hydraulic overflow rate was increased from ~2.75 to 3.75 gpm/sq ft.

	Low Overflow Rate	High Overflow Rate
BOD	59	52
COD	57	54
Suspended Solids	70	61
Volatile Suspended Solids	71	64

10. Utilizing a 50 mesh (297 μ) screen at a hydraulic loading of 40 gpm per sq ft of screen. The percent removal efficiencies for screening alone were observed to be:

BOD	27 ± 5
COD	26 ± 5
Suspended Solids	27 ± 5
Volatile Suspended Solids	34 ± 5

11. Capital cost estimated for a full scale screening/flotation system completely installed was $21,056 per MGD capacity or $3,828 per acre for a plant capacity of 90 MGD. These costs were distributed as follows: screening/flotation modules and accessories 71%; sewers and outfall 4%, land costs 9%, engineering and related costs 16%.

12. The operating costs for a screening/flotation system were estimated at 3.09¢/1000 gallons of treated overflow. These costs include chemical flocculant addition. These costs were distributed as follows: chemicals 81%, power 17%, maintenance 2%.

13. The screening/flotation system provides sufficient detention time (~ 15 min.) for effective chlorination.

14. Because of the intermittent operation and remote location of combined sewer overflow treatment systems, completely automated operation is required.

15. The use of a 50 mesh (297 μ) screen eliminated the need for bottom sludge scrapers.

16. It was not always possible to obtain optimum chemical flocculant addition due to widely varying raw waste characteristics.

17. The following conclusions were made from the chemical oxidation study:

a. Chemical oxidation of combined sewer overflow is not technically or economically feasible.
b. Ozone was the best oxidant evaluated.
c. Chlorine and hydrogen peroxide are not effective oxidants.
d. Particulate organic matter is very difficult to oxidize.
e. Ozone disinfection was found to be less reliable than chlorine for combined sewer overflows.

SECTION II

RECOMMENDATIONS

It is recommended:

1. that screening/flotation be utilized to reduce the pollution from combined sewer overflows where treatment is the preferred alternative.

2. that the design and operating procedures outlined in the conceptual design section of this report be utilized for screening/flotation systems treating combined sewer overflow.

3. to investigate the effects of the unit process of flocculation on flotation removal efficiencies.

4. to investigate methods of control of chemical addition as waste strength varies.

5. to evaluate the dewatering characteristics of the sludges produced and various sludge disposal alternatives.

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SECTION III

INTRODUCTION

During recent years combined sewer overflow has been recognized as a significant pollution problem. Immediate reaction was to separate the sewer systems. It was found, however, that this solution was expensive and sometimes impossible. Furthermore, it has also been shown that stormwater could carry a large pollutional load, therefore, tending to reduce the effectiveness of sewer separation. As research to find a solution to the combined sewer problem began, it became apparent that no one solution would solve all the problems associated with combined overflows. The ultimate solution depends upon successful application of many different approaches which may include separation of sewers, holding tanks, and treatment systems.

The objective of EPA/WQO Contract Number 14-12-40 is to develop and demonstrate a method of treating combined sewer overflows. It is envisioned that the system or systems developed during this project will be utilized to provide effective and economic treatment for those combined sewer overflows where treatment is the best alternative.

Initially, the technical approach in this project was screening and chemical oxidation of combined overflows. Early during the study it was determined that chemical oxidation was not feasible and this approach was abandoned. A summary of the work done on chemical oxidation may be seen in Appendix I. During the investigation of chemical oxidation, however, a promising alternative was uncovered. It was discovered that the pollutants in combined sewer overflow consisted mainly of particulate matter. This clearly indicated that an efficient solids/liquid separation process should provide a high degree of treatment for combined overflows. The work scope of the existing contract was modified to allow study of a screening/dissolved-air flotation system. Later, after evaluating the system for a short time, it became apparent that a more detailed study of the screening process could provide important additional information and the contract was amended to include these studies.

The amended contract is divided into the following phases.

I. Literature search, site arrangements, preliminary investigations
II. Design and construction of a 5 MGD screening/flotation system
III. Operation of the screening/flotation system
IV. Design and construction of a 5 MGD series screening system
V. Operation of the screening/flotation and series screening systems in various combinations
VI. Final report, system disassembly and restoration of demonstration site

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This report summarizes the work done to date on the above mentioned phases I, II, and III. Work on phases IV, V and VI is now underway and completion is expected by March of 1972.

SECTION IV

LITERATURE SEARCH

It is assumed that the following process elements will be utilized to accomplish the treatment of combined sewer overflows.

A. Solids/liquid separation

   1. Screening
   2. Dissolved-Air flotation

B. Disinfection of the waste flow

A search of the existing literature was made to determine the characteristics of combined sewer overflow, and to obtain pertinent information on the above mentioned process elements.

Characteristics Of Combined Sewer Overflows

More than 1400 U.S. communities serving 50 million people have what is known as combined sewer systems which provide one collection system for both sanitary sewage and stormwater runoff (1). During dry weather the flow in such a system consists mainly of sanitary sewage and is normally intercepted by the interceptor systems before it reaches the outfall, where it otherwise would be discharged to a lake or river. As long as the capacity of the interceptor system exceeds the flow in the combined sewer, the flow is directed to the sewage treatment plant for purification.

During a storm, the flow in the combined sewer can increase from 50 to 100 times the normal dry weather flow (2). Normal interceptor capacities are between 1.5 and 5.0 times the dry weather flow (hereafter abbreviated DWF)(2)(3)(4). This would mean that at normal interceptor capacities, the interceptors would be from 97.3 to 99.0% efficient in collecting and transporting the sanitary sewage to the treatment plant. These efficiencies are very misleading, since during periods of high stormwater runoff, it is possible to have up to 96% of the sanitary sewage bypassing the interceptor system and being discharged untreated into the receiving waters (5). This discharge of untreated combined overflows could cause severe pollution problems if the water is reused a relatively short distance downstream as a water supply or for recreational activities.

The pollution caused by combined sewer discharge is directly related to the frequency of storm runoff exceeding the interceptor capacity. Three independent investigators (5)(6)(7) report that the normal dry weather flow is approximately equivalent to a rainfall intensity of about 0.01 inch per hour. With interceptor capacities of 1.5 to 5 times the dry weather flow, rainfall intensities greater than 0.015 to 0.05

inches per hour would be necessary to cause combined overflow which would go untreated. Palmer (6), Johnson (7), and McKee (5) studied the rainfall records of Detroit, Washington, D.C., and Boston. The results of their findings showing the number of combined overflows per month for given interceptor capacities may be seen in Figure 1. With an interceptor capacity of 5 times (DWF), the number of overflows per month was between four (4) and six (6). This indicates the extent of possible pollution of the receiving body of water.

The pollutional effect of combined sewer overflows will, of course, be determined not only by the frequency of overflows, but also by the quality of the overflow. The most important and commonly discussed pollutional parameters associated with combined sewer overflows are suspended solids, BOD, and coliform count. To establish a basis for comparison, the average raw (primarily domestic) sewage contains 200 to 300 mg/l suspended solids (1)(8)(9), (50 to 60% of these suspended solids are settleable), 100 to 300 mg/l BOD (1)(8)(9) and coliform counts of 43 to 150 x 10^4 per ml (9)(10).

Assuming pure stormwater to be essentially unpolluted, it would be expected that the BOD, suspended solids, and coliform counts of combined sewer overflow to be substantially lower than those of raw sewage. This assumption could result in considerable error when estimating the quality of combined sewer overflows. A limited amount of data has been obtained which indicates the quality of pure storm runoff is highly variable and can be quite polluted (4)(6)(11)(12)(13)(14). BOD's ranging from 6 to 600 mg/l, suspended solids ranging from 100 to 2000 mg/l, and coliform counts as high as 200,000 per ml have been reported.

Another factor which can greatly influence the quality of combined sewer overflow is the amount of solids which settle in the sewer system during periods of low flow. Since combined sewer systems must be designed for a wide range of flow, the amount of solids which are deposited during periods of dry weather can be significant (15)(16)(17). Hopefully, these solids will be resuspended during the first flushes of the storm and be intercepted before combined overflow begins. Published data has shown, however, that with an interceptor capacity of only 1.5 to 5 times the dry weather flow and a sewer system designed to take 50 to 100 times the dry weather flow, most of the solids which have settled are not resuspended until after the capacity of the interceptors has been exceeded (11)(14)(15)(16)(17). Hence, these solids can contribute greatly to the pollutional load in the stormwater overflow.

The above discussion has illustrated that combined sewer overflows can contribute a significant pollutional load to receiving streams or lakes. Table 1 summarizes data reported by others on the quality of combined sewer overflows. It is obvious that the quality of combined overflows is highly variable. Of the three pollutional parameters most mentioned, i.e., BOD, suspended solids and coliform count, the coliform count, a measure of pathogenic bacteria contamination (8), appears to be the most significant (10)(13). In a recent study (13), a survey was made to indicate what water uses were most affected by combined

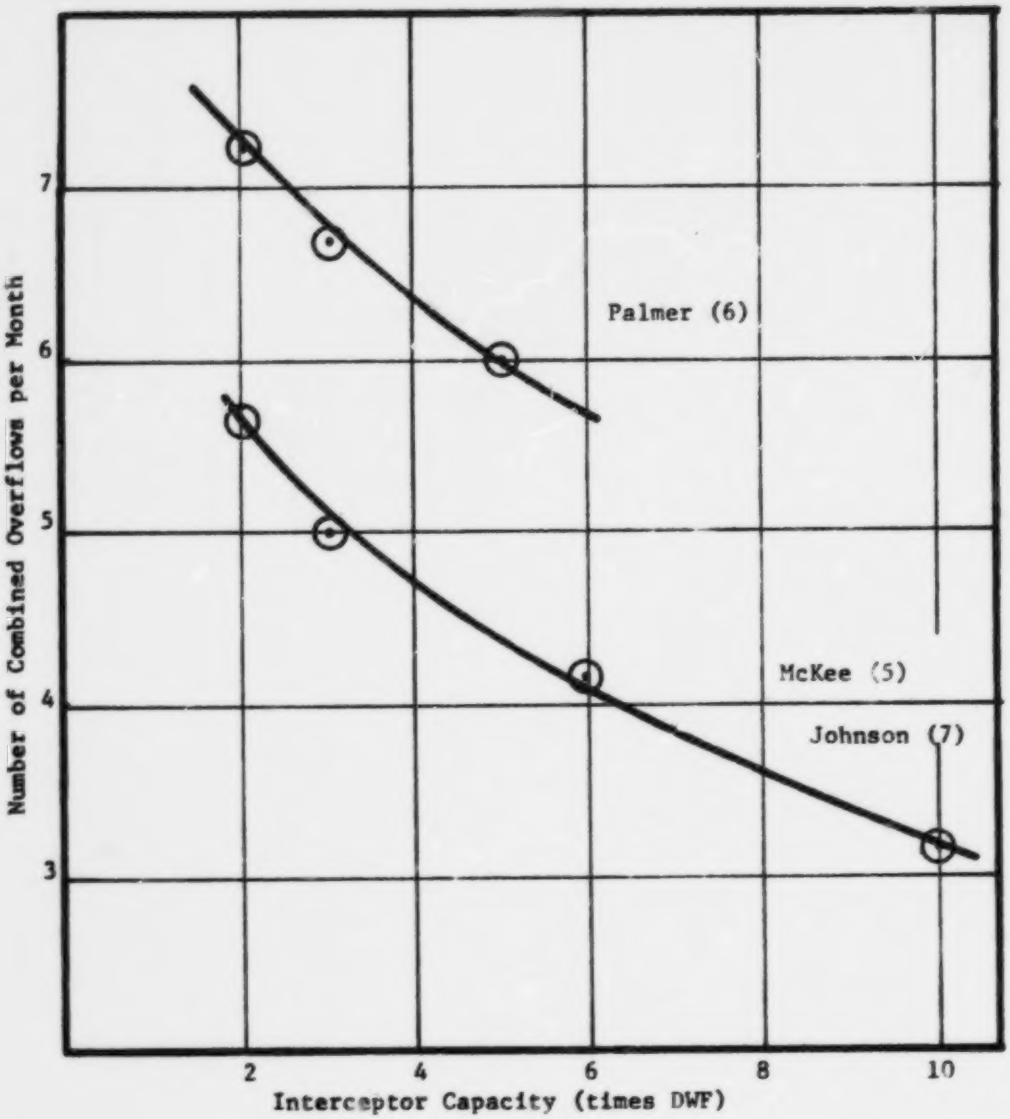


FIGURE 1

RELATIONSHIP OF FREQUENCY OF
COMBINED SEWER OVERFLOWS TO
INTERCEPTOR CAPACITY

TABLE 1

QUALITY OF COMBINED SEWER OVERFLOWS

Source Reference Number	BOD mg/1	Suspended Solids mg/1	E. Coli Density per ml.	Discussion
(6)	50	250	43,000	Detroit, Michigan Average Estimated Values
(14)	100	544	--	Buffalo, New York (Bird Avenue) 1 Sample
(13)	59	203	300,000	San Francisco, California (Average Value 14 Samples)
(13)	92	129	500	Chicago, Illinois Average 31 Overflows (1962
(13)	121	436	--	Buffalo, New York (Bailey Avenue)
(12)	--	150	180,000	Average Values, 43 Samples 1964 Data
(11)	40-260	130-930	2300-24000	Toronto, Ontario, Canada (Eglinton Avenue)
(11)	220-614	168-426	2,100	Welland, Ontario, Canada (Burgar Street)

overflows. The survey indicated those uses most affected were commercial (fishing), recreational (swimming), and public health (water supplies). The main pollutant affecting these water uses is pathogenic bacterial contamination (measured by coliform density) (1)(5)(15)(16). Table 1 shows the range of coliform counts to vary from 0.5×10^3 to 300×10^3/ml in combined sewer overflows. (A review of disinfection starts page 17.)

The suspended solids concentration of combined overflow can be quite large. Table 1 shows a range of suspended solids in combined overflows of 129 to 390 mg/l. The values shown in Table 1 are averages. The initial peak of suspended solids can be higher (11)(15)(14). Romer and Klashman (16) presented graphs which illustrate how suspended solids in combined overflows vary as the flow increases. They reported that, in general, the suspended solids concentration and the flow reach a maximum value at approximately the same time. The suspended solids then begin to fall off rapidly. Stegmaier (17) has shown that the main increase in solids is of inorganic nature. Initially, the BOD exerted by the solids will probably be quite large, since the settled organic material will be resuspended before the inorganics. Hence, the BOD of combined sewer overflows will initially be quite high with a large portion of the BOD in the particulate form. This would seem to indicate some sort of mechanical separation may be very effective in removing BOD during the first flushes of a storm. Later, as the storm progresses the BOD would be expected to decrease to levels much lower than those of raw sewage. The ratio of dissolved to particulate BOD will increase and mechanical separation will probably become less efficient in removing BOD.

Removal of Solids by Screening

Bar screens with openings of 1/2 to 1-1/2 inches are used extensively in sewage treatment plants throughout the United States (18). The amount of solids removed by these coarse type screens ranges from 0.5 to 6 cubic feet of wet solids per million gallons of sewage (19)(20). On a dry weight basis, this represents 9 to 48 pounds of dry solids per million gallons of sewage (19)(20) or a removal of about 1 to 6 ppm from the sewage flow. Since raw sewage contains 200 to 300 ppm (1)(8)(9) suspended solids, bar screens remove only a small fraction of the solids, i.e., 0.5 go 2%. This fact is not surprising, since the primary purpose of bar screens is to remove large and/or floating solids to protect pumps and other treatment devices downstream of the bar screens (8)(18).

In general, the approach velocity in the screening channel should not fall below a self-cleaning value (about 1.25 fps) or should it rise to values high enough to force the screenings through the bars (about 3 fps) (8)(18). Headlosses through bar racks can be formulated as an orifice loss (8).

$$h = B(w/b)^{4/3} h_u \sin\theta$$

where

h = loss of head in feet
w = maximum width of the bars facing the flow
b = minimum width of the clear openings between pairs of bars
h_u = velocity head (in feet) of the water as it approached the rack
θ = the angle of the rack with the horizontal
β = bar shape factor

Values for β are 2.42 for sharp-edged rectangular bars, 1.79 for circular bars, and 0.76 for tear-drop shaped bars (8). Headloss through bar racks is generally held below one foot by periodic cleaning of the bars. This cleaning can be accomplished manually or mechanically (8).

Fine screens 1/16 inch to 1/32 inch openings have been used in a number of locations (19)(20)(21)(22) to treat municipal wastes. This size of screen will remove 2 to 20% of the suspended solids in raw sewage (8)(18)(19)(20)(21)(22). Openings of 1/16 inch, correspond to a sieve size (Tyler series) of 12 to 20 mesh. Use of standard wire sizes finer than these meshes tend to blind because of the build-up of animal fats and greases on the screen (23). It has been reported by Peterson (23), however, that the use of new synthetic materials for screens has greatly reduced the blinding tendency and allows greases to be discharged from the top of the screens. He also reports meshes as fine as 300 have been used when filtering raw sewage. The allowable rates of filtration, with these extremely fine mesh screens, however, were not reported. The results of screening raw sewage through a 300 mesh screen showed a reduction in suspended solids from 196 to 101 mg/l or a 48.5% removal (23). This removal rate is approximately equal to the rates obtained by primary sedimentation of raw sewage (8) (18).

As the openings in the screen are reduced, the headloss increases and screen blinding becomes very important. This necessitates automatic mechanical cleaning, which is generally accomplished by backwashing the screen with jets of water and/or air. Headlosses through fine screens can be calculated from an orifice type equation.

$$Q = CA\sqrt{2gh}$$

where:

Q = flow (cfs)
A = area (sq ft)(actual open area)
g = gravity constant (32 ft/sec^2)
C = screen coefficient
h = headloss through the screen (feet)

Screen coefficients (C) vary with the size of the screen openings. Values are generally between 0.3 to 0.6 for 2 to 60 mesh screens.

Headlosses through fine mesh screens are generally held below 2.5 feet (8) by varying the frequency of cleaning.

There is a very limited mention found in the literature on screening of combined sewer overflows. Stegmaier (17) and Romer and Klashman (16), however, have shown that a large portion of the suspended solids in combined sewer overflows can be attributed to inorganic materials. This tends to indicate that fine screening of combined sewer overflows may show a higher percentage removal of suspended solids than is obtainable when screening raw sewage. Boucher and Evans (24) have discussed the microstraining process and its applications in removal of suspended material. They have indicated that microstraining can be effectively and economically used in polishing of sewage effluents. Metropolitan Sanitary District of Chicago is reported to be utilizing microstrainers for its Hanover Water Reclamation Facility (25). In a recently reported study by Keilbaugh et al (26), up to 98% of suspended solids were removed from a combined sewer overflow by using a nominal 23 micron aperture microstrainer screen. However, most of the data published towards the use of microstrainers in waste treatment has been of investigational nature only, and significantly more data is needed to substantiate such findings.

Dissolved-Air Flotation

An active use of dissolved-air flotation (DAF) in the waste treatment area has only been made in the last two decades, but the use of air-bubbles to change the apparent specific gravity of suspended matter in water has been practiced for nearly a century in the mining industry for the beneficiation of ores. Selective adsorption was accomplished by the use of flotation agents. Chase (27) and Rohlich (28) have discussed the developmental background of dissolved-air flotation. The process as it is used now is an outgrowth of air flotation methods used in the recovery of paper fibers. The fiber recovery was achieved by the adsorption of an air bubble on the fiber. The floated material formed a scum and was collected by a suitable mechanism. The presence of alum and addition of glues aided the separation of fiber by the flotation process. Dissolved-air flotation utilizes a much smaller diameter bubble (less than 100 microns) than that used in the mining industry. It is clear from the above discussion that the removal of suspended matter is accomplished when there exists sufficient adsorption forces between the air bubble and solid particle.

Geinopolos and Katz (29) have discussed solids-liquid separation by dissolved-air flotation as compared to sedimentation and have indicated that a waste could be treatable by either sedimentation or flotation, but higher separation rates and solids concentration may be possible by dissolved-air flotation. This results in smaller basins, smaller sludge volumes and higher water recoveries for treatment by flotation processes as compared to sedimentation. The mechanism and driving forces involved in the flotation process is similar to those encountered in sedimentation. Stoke's Law illustrates the mechanism by the following equation:

$$V = gD^2(\gamma_s - \gamma_l) / 18\mu$$

Where:

V = Particle separation velocity, ft/sec
g = Gravity constant, ft/sec^2
D = Diameter of particle, ft
γ_s = Density of particle, lb/cu ft
γ_l = Density of liquid, lb/cu ft
μ = Viscosity of liquid, lb/ft/sec

In the flotation process, the density of the air-solid combination is less than the suspending medium and value of V becomes negative, causing an upward particle velocity. Also, the effect of the air-bubble is to increase the difference in densities between the particle and suspended medium, which in turn increases the solids separation rate.

Van Vuuren et al (30) has reported the use of oxygen produced by algal photosynthesis to accomplish flotation. He also cites the applicability of dispersed air flotation in conjunction with chemical precipitation in advanced treatment of wastes (31). Chase (27) has described the types of dissolved-air flotation processes used for waste treatment. These include pressure and vacuum types. However, pressure type applications in waste handling far outnumber those for vacuum flotation, primarily because of inherent flexibility in a pressurized system. The vacuum process is limited by the amount of reduced pressure that can be effectively used. Geinopolos and Katz (29) state that the process elements of dissolved-air flotation are:

(1) Flow pressurization
(2) Air Introduction
(3) Air Solution
(4) Pressure reduction and bubble formation
(5) Bubble/solids attachment
(6) Solids/liquid separation
(7) Separated solids removal

The literature describes variations in the flow pressurizing process (27)(28)(32)(33). These include: total pressurization, in which the entire volume of raw waste is pressurized, split flow, in which a portion of the raw waste is pressurized and later blended with the remaining raw waste stream; and effluent pressurization, in which a portion of basin effluent is pressurized and later blended with the entire volume of raw waste.

The dissolved-air flotation process has been applied to remove suspended matter from many industrial wastes such as: paper wastes (35), refinery wastes (28)(36), laundry wastes (37), soap wastes (38), machine shop wastes (39), automobile wastes (40), and others (27)(28) prior to discharge in a receiving body of water or reuse. Vrablic (34) in his discussion of the fundamental principles of dissolved-air flotation mentions that hydrophobic solids will float much more easily than will hydrophilic ones. In an investigation of the kinetics of removal of organic matter by activated sludge, Rohlich and Katz (4) demonstrated that activated sludge floc can be separated from water by dissolved-air flotation. Ettelt (42) has also shown that dissolved-air flotation can be successfully employed for activated sludge thickening. There have been several other reports on the use of dissolved-air flotation for dewatering of aerobic biological solids (43)(44)(45)(46).

Proper performance of a flotation unit is dependent upon sufficient air bubble/solids attachment to effect good flotation. Batch laboratory flotation tests as described by Eckenfelder (47) have been used to estimate the flotation characteristics of wastes for the purposes of design. Howe (48), in a mathematical derivation of flotation cell design also recommended that considerable experimentation with each different waste precede the use of his equations in determining the exact criteria for flotation cells. For sludge thickening application, air/solids ratio has been used to relate laboratory data to prototype facilities. A ratio of 0.03 to 0.1 is considered optimum for most sludges (34)(49). However, air/solids ratio for clarification alone is generally not critical. Truck mounted continuous flow, pilot scale units are used widely for obtaining design information. The primary variables for flotation design are operating pressure, ratio of pressurized flow to raw waste flow, retention period, and the combined particle/air bubble rise rate. Generally, the effluent suspended solids decrease and the concentration of solids in the floated sludge increase with increasing retention period. For clarification, a detention period of 15 to 30 minutes is normally considered adequate in the flotation zone. Rise rates of 1.0 to 3.0 gpm/sq ft (0.13-0.39 feet per minute) are commonly employed. The amount of pressurized flow usually varies from 15 to 50% of raw waste flow for separation. The recycle amount and detention period are higher for thickening application depending upon feed solids concentration. The process pressure may vary from 30 to 70 psig, with 40 psig considered normal. Air requirements range from 0.5 to 1.0 cu ft of standard air per one hundred gallons of pressurized flow (32). The air and liquid are mixed under pressure in a retention tank with a detention time of 1 to 3 minutes.

Addition of chemicals may be employed depending upon the nature of the waste and the degree of treatment required. Chemicals such as alum, lime and ferric chloride are usually used in the chemical treatment of a wastewater. The purpose of chemical treatment is to precipitate and coagulate colloidal and finely suspended particles to form large suspended floc particles. This increases the separation rate of the solids by flotation. Recent efforts to improve flotation efficiency by chemical addition have involved use of synthetic organic polyelectrolytes. Generally, these polymers have been found to improve solids capture (clarification), to increase the concentration of solids in the float and to increase the capacity of flotation equipment (50)(51)(52). An apparent advantage of polyelectrolytes is the elimination of a separate flocculation tank, since the floc particles are formed in an extremely short period of time. Cationic polyelectrolytes have been successfully employed as flotation aids for thickening applications (46)(53). The results indicate that desired float concentrations may be achieved with high solids loadings when polymers are utilized. Bench scale procedures are normally employed for determining the dosage and effectiveness of a particular polymer application in dissolved-air flotation.

Disinfection Of Combined Sewer Overflows

Disinfection may be defined as destroying those bacteria that cause infection or disease. Sterilization is defined as the destruction of all living organisms (54). Methods commonly used for disinfection include heat, ultraviolet light, and chemical addition. Chemicals which have been used include chlorine, bromine, iodine, potassium permanganate and ozone (8). The various methods and chemicals used for disinfection which appear to be applicable to this project will be discussed below.

Ultraviolet Light Disinfection

The effect of radiation on bacteria has been studied in detail, and the relationship between wave length and germicidal effect is well known. The optimum wave length is 2600 angstrom, hence, low pressure mercury vapor lamps with their high output at 2537 angstrom are effective bactericidal agents (55). Light of this wave length falls in the ultraviolet section of the light spectrum. To insure disinfection, the water should be relatively free of suspended matter which might shade the organisms against the light (8). Time and exposure intensity must also be adequate to insure disinfection. One hundred microwatts per square centimeter of 2537 angstrom light will produce high bacteria kills at contact times of less than one minute (55)(56).

Use of ultraviolet light for disinfection has found limited application, probably because other methods of disinfection are more economical (8).

Disinfection By Chemicals

Chlorine and chlorine compounds have proven so economical and efficient that they are widely employed for their bactericidal action. The chlorination of wastes may be regulated to accomplish various degrees of bactericidal action. Chlorine is a bacteriostatic agent when applied in small concentrations, to prevent an increase in the bacterial population. Chlorine is a disinfecting agent when applied in larger amounts to destroy those bacteria that cause infection or disease. Chlorine may be, but seldom is, applied in amounts so large as to be a sterilizing agent, i.e., to destroy all living organisms (54).

Forms of chlorine which may be used for disinfection and other purposes include gaseous or liquid chlorine, chlorinated lime, calcium hypochlorite, sodium hypochlorite, chlorinated copperas, and chloramines. Chloramines, however, are a slower acting and less active form of disinfectant. Liquid or gaseous chlorine is the most commonly utilized form.

The efficiency of disinfection using chlorine or other disinfecting agents is dependent on (a) the contact time, (b) the type and concentration of microorganisms, (c) the pH and temperature of the water, (d) the presence of interfering substances, (e) and the degree of protection afforded organisms from the disinfecting solution by materials in which they may be imbedded.

Generally, a chlorine dosage, sufficient to give a 0.5 ppm residual after a 15 minute contact time, is considered adequate for disinfection (54). The actual amount of chlorine added to obtain this residual will vary with the waste being chlorinated. Fair and Geyer (8) give values of 6 to 24 mg/l for raw sewage.

Camp (10) indicates screening and chlorination of combined sewer overflows would be an effective and inexpensive way of treating these flows. Chlorine dosages were estimated to range from 1.6 to 8.5 ppm.

Symons (57) found the chlorine demand for Buffalo sewage during periods of combined overflow to range from 6 to 7 mg/l. Considering the possible quantities of combined sewer overflow, this represents quite a substantial chlorine dosage, and will require chlorine feeders operating over a very wide range of flows.

Iodine and bromine were also found to be effective in disinfecting sewage, but required higher dosages compared to chlorine (58). Fluorine was found to be so reactive that it was difficult to store and apply (58).

Potassium permanganate has been used to disinfect water supplies (59) (60), and is an effective disinfectant at relatively low dosages (59). Use of potassium permanganate has not been widespread. This is probably due to one or more of the following disadvantages of potassium permanganate disinfection when compared to chlorine disinfection.

1. $KMNO_4$ imparts a color to the water which must be removed before use (62).

2. Most effective pH for $KMNO_4$ is 5.9 which is below the pH of many water sources.

3. Potassium permanganate is not a good post-treatment disinfectant because of the color and pH criteria.

Ozone is a very effective disinfectant. This fact is well known and has been proven over a period of 60 to 80 years (35)(46)(61)(62)(63). Ozone is used extensively in Europe, especially in France (62). Disinfection by ozone follows a slightly different pattern when compared to chlorine. Generally, with increasing chlorine dosage the number of bacteria progressively decrease, while with ozone little reduction is noticed until a critical dosage is reached. At this critical value, the bacterial population is rapidly reduced (35).

Unlike chlorine, the disinfecting action of ozone is little affected by changes in temperature or pH. Ozone acts rapidly and is almost instantaneous, where chlorine needs time to be effective (35)(61).

The reason ozone is not utilized to the same extent as chlorine is probably twofold, i.e., cost and inability to carry an ozone residual for any length of time. Hann (62) states that, in general, if disinfection is the sole objective, ozone is more expensive than chlorine. When using ozone as a disinfectant, it is not possible to carry an ozone residual for a significant period of time (35), and this is sometimes cited as a disadvantage when using ozone.

When using ozone proper precautions must be taken in applying the ozone to insure disinfection (62). The water should be low in turbidity, the ozone demand must be satisfied, and there must be a residual of free ozone in the water for a definite period of time. This usually is from 0.1 to 0.2 ppm ozone residual for 1 to 5 minutes (35)(62).

Summary Remarks

The material contained in this literature search was compiled in an effort to uncover the known process elements applicable to the treatment of wastes similar to combined sewer overflows. The information gained from this literature search will be used as a guideline for laboratory and bench scale tests to determine what combination(s) of the various process elements is (are) best suited to the design, fabrication, and operation of a prototype combined sewer overflow treatment unit.

Although a literature search is technically concerned only with the reporting of data found in published form, some of this data, for one reason or another, can be eliminated from further consideration without the benefit of laboratory testing. As a result, this report comprises a certain number of engineering judgments and/or evaluations. Such judgments, however, have been made only where the material available or the investigator's experience have justified making them. That data which appeared to have genuine value to the completion of this project has been carried forward to the laboratory studies portion of the project.

Significant observations from the literature search are summarized below:

1. The flow in combined sewers can increase 50 to 100 times the dry weather flow.

2. Normal interceptor capacities are between 1.5 and 5.0 times the dry weather flow.

3. Up to 96% of the sanitary sewage can bypass the interceptor system during heavy storms.

4. Rainfall intensity greater than 0.015 to 0.05 inches per hour is generally sufficient to cause combined sewer overflow.

5. Limited data indicate pure storm water can be quite polluted.

6. The majority of the solids settled in the sewer system during dry weather are not resuspended until the capacity of the interceptor system is exceeded.

7. It was indicated that the main water usages effected by combined sewer overflow were commercial fishing, swimming, and public water supply due to bacterial contamination.

SECTION V

SITE SELECTION AND PRELIMINARY INVESTIGATIONS

Site Selection

A search was conducted to select a suitable demonstration site for the installation and operation of a demonstration treatment unit. The City of Milwaukee, as an available source of combined sewer overflow, was solicited in regard to participating in the demonstration of the proposed treatment system. The City of Milwaukee responded favorably by offering both access to a combined sewer as well as providing sufficient land to install the demonstration equipment. In addition, the City Engineer's Office was made available for consultation and aid in selecting the most beneficial demonstration site.

Six potential sites were investigated. These sites were located in the City of Milwaukee as follows:

1. The Edgewood Avenue site, consisting of a 72 inch combined dewer discharging into the Milwaukee River.
2. The 27th Street site (south end of viaduct), consisting of a 48 inch combined sewer discharging into the Menomonee River.
3. The East Kane Place site, consisting of a 72 inch combined sewer discharging into the Milwaukee River.
4. The Bay Street site, consisting of a 90 inch diameter combined sewer discharging into Lake Michigan.
5. The Russell Avenue site, consisting of a 120 inch diameter combined sewer discharging into Lake Michigan.
6. The Hawley Road site, consisting of an 8'6" by 5'0" combined box sewer discharging into the Menomonee River.

The potential sites were appraised and evaluated from the standpoint of availability for project duration, access to sewer flow, access to utility sources, space limitations, extent of structural and/or topographical modifications, proximity to residential or developed areas, drainage area covered, and the socio-economic nature of the drainage area.

By inspection, the Edgewood Avenue, 27th Street, and East Kane Place locations were eliminated as potential demonstration sites. The Edgewood Avenue and 27th Street sites were inaccessible and were limited in space. The East Kane Place site was located in a residential and developed area. In fact, the site consisted of a city-owned lot, located between two residences, which were about thirty feet apart.

The Bay Street, Russell Avenue, and Hawley Road locations were further investigated as potential demonstration sites. Maps of the respective combined sewer areas were obtained from the City Engineer's Office in which the drainage area covered and the socioeconomic nature of the drainage area were delineated. The areas drained by each of these three sites is predominantly residential with some local business and industrial activity. The drainage areas covered by the three potential sites were as follows:

Bay Street	385 acres
Russell Avenue	465 acres
Hawley Road	495 acres

The Bay Street location was eliminated because an investigation revealed that it would be difficult and impractical to impound the flow in the sewer to provide the necessary raw sewage pump suction conditions.

The Russell Avenue site was eliminated because of a restrictive time limit placed by Milwaukee County authorities upon the use of the site for project purposes. It was felt that the site may not be available for the full duration of the project.

Since the Hawley Road site appeared to be the most desirable locatation for a demonstration unit, additional information was obtained on this sewer and the area served.

As mentioned above, the total area served by this sewer is 495 acres. The area is located on the western edge of the Milwaukee City limits. Figure 2 presents a street map of the City of Milwaukee. The outlined portion depicts the drainage area served by the Hawley Road sewer. The drainage area was studied using recent (1970) aerial photographs. It was determined that approximately 42% of the area was impervious, i.e., roof tops, streets and parking lots. This value is within the range presented in the literature (8) for North American cities in areas of 1 and 2 family dwellings. The relationship between runoff coefficient and perviousness is presented in Figure 3. The data in Figure 3 is based on the City of Milwaukee runoff curves. It may be seen from Figure 3 that the runoff coefficient for the Hawley Road drainage area is 0.40. While it is recognized that the runoff coefficient can vary as the storm progresses (8), no attempt was made to refine the runoff coefficient, since an exact number was not critical to this project. The aerial photograph indicated that the area is a completely developed residential area of one of the older sections of the city. There is very little open area present such as parks or fields. All dwellings are of the one and two family variety. Population density based on number of dwellings per acre and assuming four persons per family was estimated at 35 persons per acre. There are some small

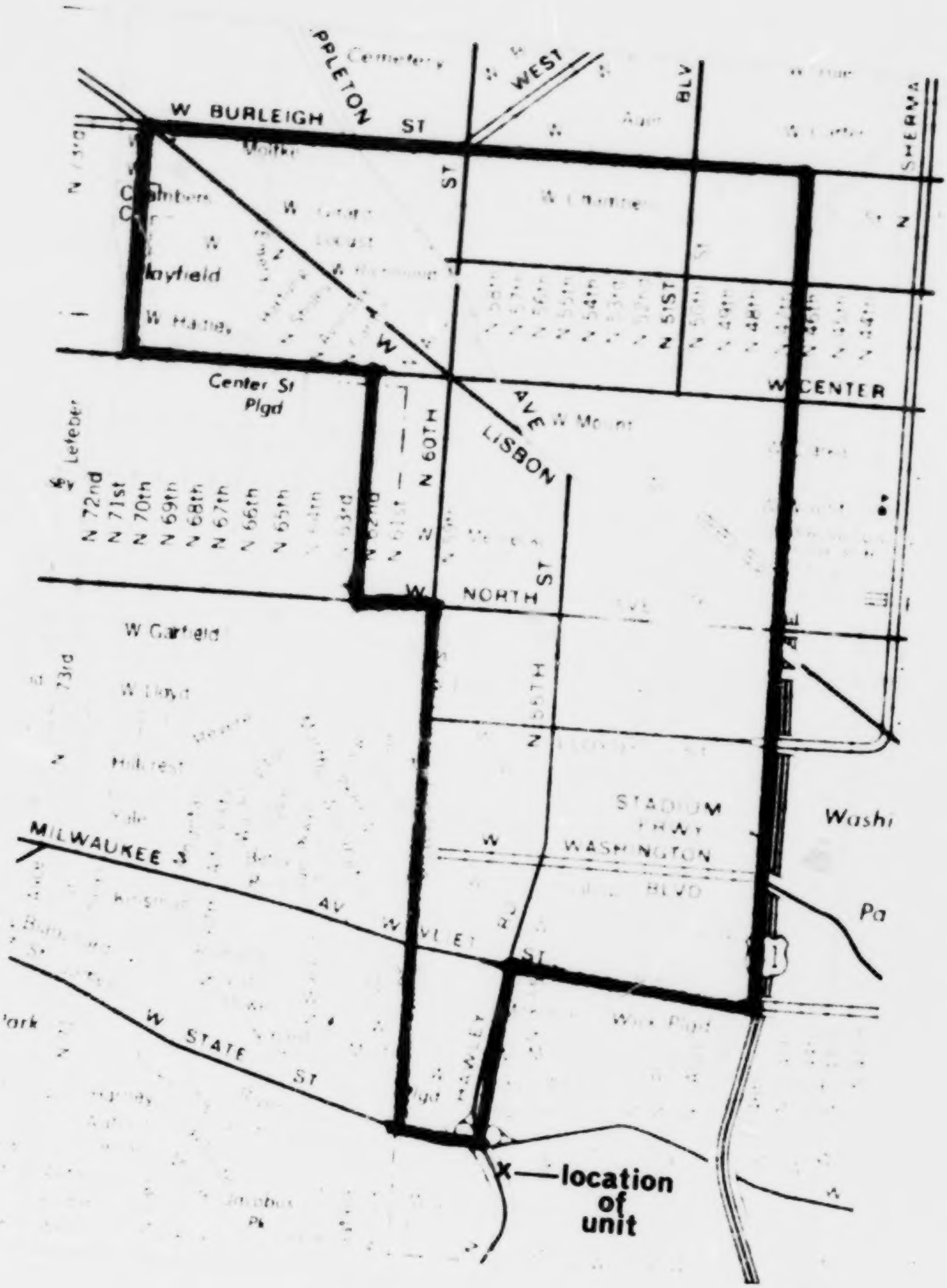
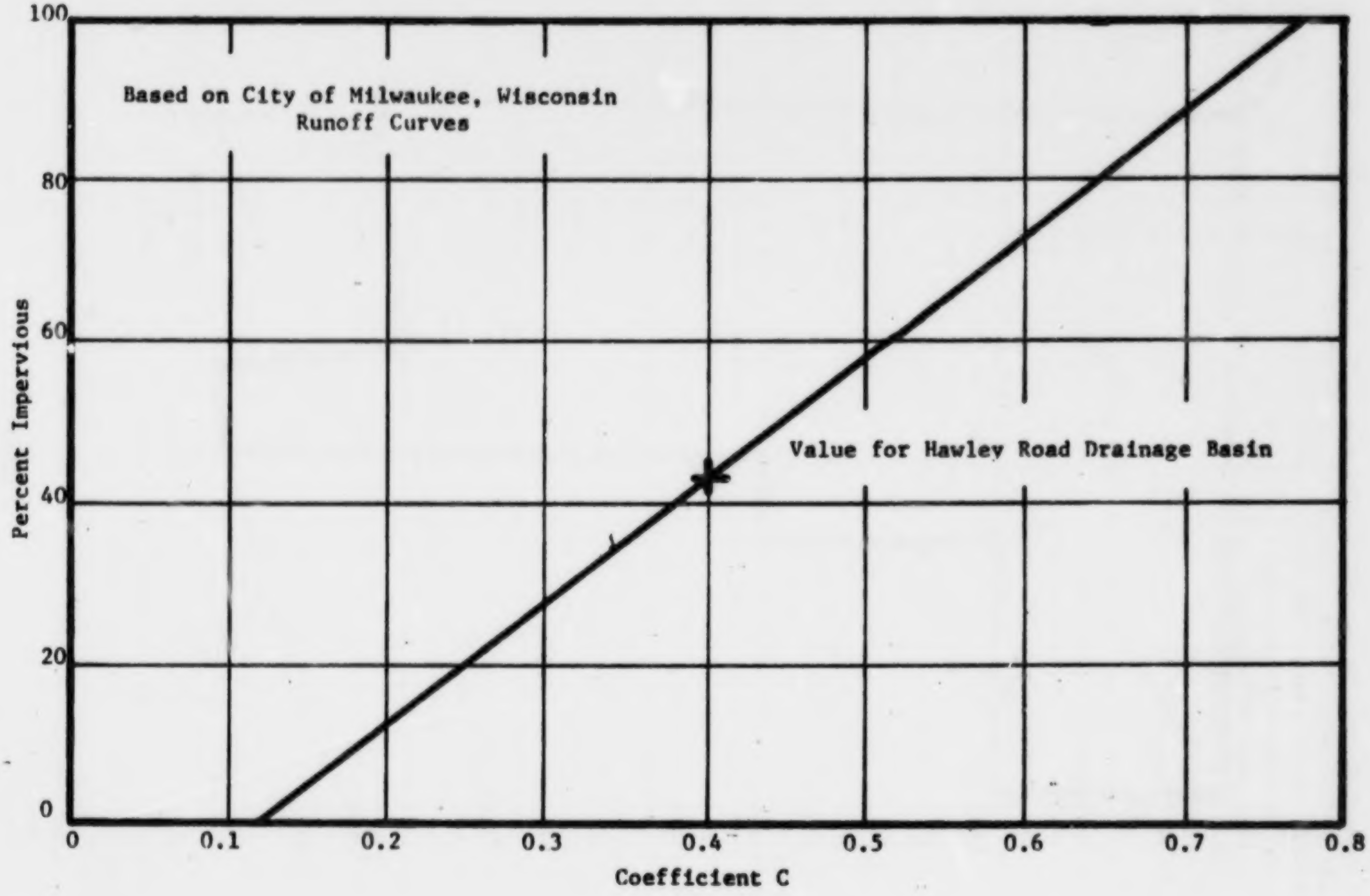


FIGURE 2

PROJECT DRAINAGE AREA

Reproduced from best available copy.


Based on City of Milwaukee, Wisconsin
Runoff Curves
Value for Hawley Road Drainage Basin
Percent Impervious
100
80
60
40
20
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
Coefficient C

shops such as jewelry, hardware, etc. No industrial manufacturing facilities are located within the drainage area served by the Hawley Road sewer. The one industry which is within the drainage area is Topp Oil and Supply Company. They are involved with blending various oils and apparently do not actually process any oil at this site. A discussion with personnel at the Milwaukee Sewage Treatment Plant revealed that Topp Oil did not discharge any significant pollutional load into the sewage system.

The combined sewer outfall serving the Hawley Road drainage area is rectangular in cross section 8.5 feet wide and 5 feet high. The relationship between water depth and flow rate is shown in Figure 4. It may be seen from the figure that the sewer has a capacity well in excess of 100 MGD. Because of the location of the sewer and the rectangular shape, a retaining structure was easily added to impound the flow. This allowed long test runs even during periods of low rainfall intensity. Calculations show that if a 3 foot high retaining structure is placed at the combined sewer outfall to retain the flow, a volume of about 100,000 gallons can be retained for evaluation after normal overflow has stopped. The retaining structure is a movable dam which tips up as a result of water pressure if the sewer begins to surcharge. In the normal untipped position the dam does not seal the sewer completely and this insures that the sewer will always be dry prior to any overflow. At a treatment rate of 5 MGD, this reservoir would allow an additional one half hour operation of the demonstration system. In actual operation, an additional operating time of only 15-20 minutes has been realized. It should be noted that the length of operation reported herein includes this 15-20 minute storage factor and hence the duration of overflow is shorter than the length of run by this time period. The Hawley Road Sewer is fed by two interceptor devices. These interceptor devices are of the sump type. A sketch of the interceptor device utilized is shown in Figure 5.

Since the Hawley Road Sewer and adjacent land met all the requirements necessary to successfully complete the demonstration phase of this contract, arrangements were completed with the City of Milwaukee in February of 1968 to allow use of the sewer and land for this project.

Preliminary Investigations

After the site selection was finalized, 14 overflows were monitored to obtain information on overflow characteristics, and to provide raw overflow for subsequent laboratory testing. Results of the laboratory testing program were then utilized to design the demonstration system. Table 2 presents a summary of the data collected prior to system design. As was expected, the range of values is quite large. Suspended solids values from 2158 to 65 mg/l were observed, while COD values ranged between 1410 and 52 mg/l. The dissolved organic fraction of the overflows as measured by COD was only 4 to 25% of the total organic load present in the combined overflow. This indicated that an

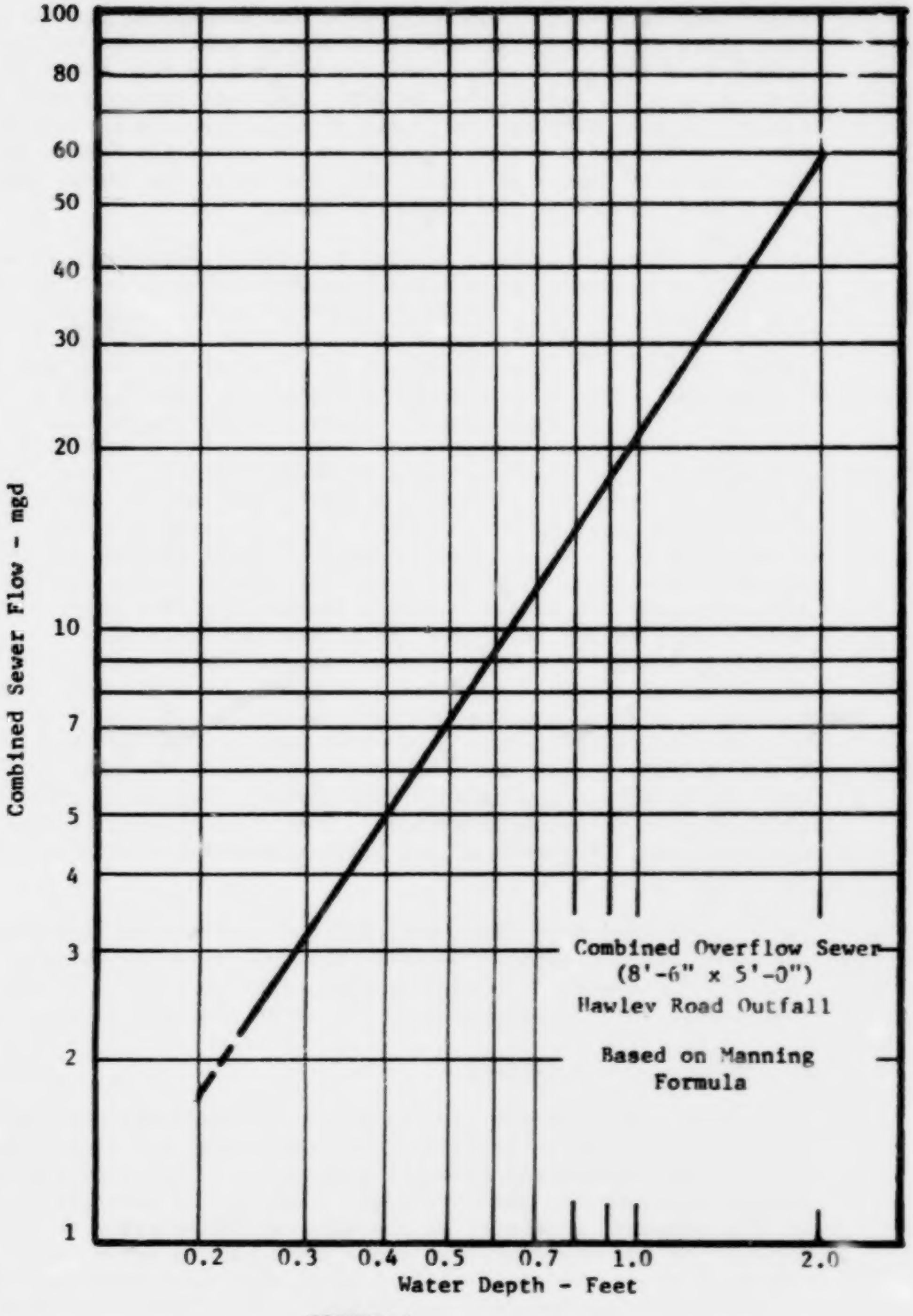


FIGURE 4

RELATION BETWEEN WATER DEPTH AND FLOW IN HAWLEY ROAD SEWER

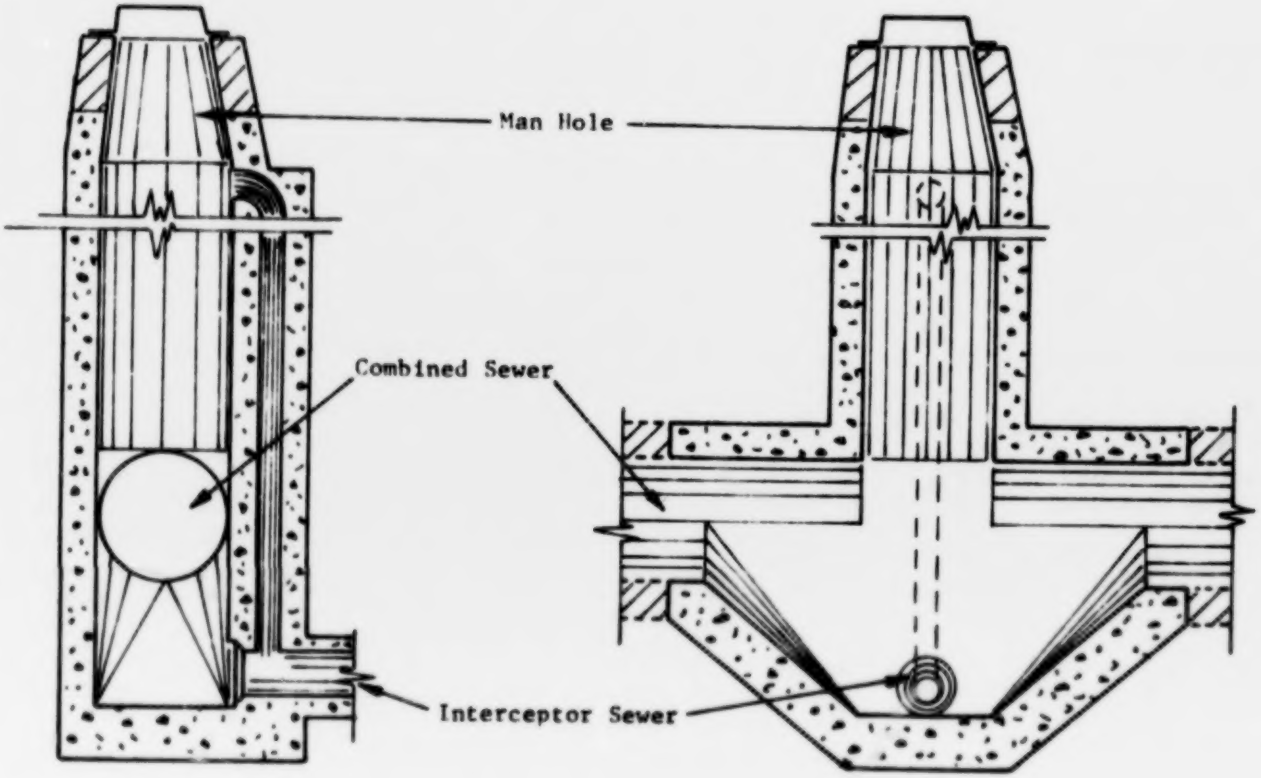


FIGURE 5

TYPICAL INTERCEPTOR DEVICE

TABLE 2

CHARACTERISTICS OF COMBINED OVERFLOW
(Fall 1967 - Spring 1968)

Date of Overflow	Total Solids mg/1	Suspended Solids mg/1	Volatile Suspended Solids mg/1	Total COD mg/1	Dissolved COD (1) mg/1	BOD (5) mg/1	Coliform #/ml	Remarks
09/27/67	388	65	---	91	---	---	440	Extended Rain
11/02/67	---	---	---	65	---	---	5000	Extended Rain-- 4 Days
11/10/67	651	418	---	520	110	---	---	First Flushes
11/16/67	664	---	---	210	---	---	---	
11/25/67	649	138	---	159	---	59	4260	First Flushes-- L. Rain
12/07/67	---	232	113	298	50	---	2151	End of Overflow
12/21/67	---	242	---	150	30	17	1860	
01/29/68	---	2158	---	1410	60	---	5730	
04/03/68	---	808	---	889	95	---	1388	First Flushes
04/03/68	---	194	---	188	48	---	421	Extended Overflow
04/17/68	---	228	---	178	---	47	1280	
04/20/68	---	70	---	52	---	13	1850	
04/23/68	---	137	---	134	---	26	6000	
04/28/68	---	461	---	476	---	111	32000	First Flushes
04/28/68	---	248	---	311	---	65	26000	Extended Overflow
05/20/68	---	194	---	251	---	54	25600	

(1) See Appendix II Analytical Procedures

efficient solids/liquid separation system should provide high removals of pollutants from the raw waste. Coliform density also varied widely from 440 to 26,000 per milliliter.

Laboratory testing which was performed included screening, chemical oxidation, flotation and disinfection. Chemical oxidation proved impractical for use in treating combined sewer overflow. Complete details of oxidants utilized, reaction times and efficiency may be seen in Appendix I.

A summary of the preliminary screening data is presented in Table 3. Various mesh sizes from 50 to 400 (297-37 μ openings) were investigated. The majority of the tests were run on a 50 mesh-297 μ opening screen. The laboratory screen test consisted of pouring the sample through the screen into a beaker. The sample was poured gently to avoid breakup of any particles, and in all tests a mat of solids was not allowed to form on the screen. This eliminated the filtering action which could have resulted if a layer of solids was allowed to form on the screen. It was anticipated that in actual field operation the screens would operate partially blinded and, hence the removal rates would probably be higher in the field when compared to the laboratory analysis shown in Table 3. All screen mesh sizes mentioned in this report are Tyler series mesh. Relationship of opening size to mesh size is presented in Figure 6.

Based on the data of Table 3, average removal of suspended solids was 18.6% for a 50 mesh screen, 32% for 100 to 200 mesh screen, and 51% for a 400 mesh screen. An exception to these figures may be seen in the data from 12/21/67 in Table 3. For this test a 120 mesh screen was utilized and the raw waste was flocculated with 1 mg/l of Reten A-1 polyelectrolyte. Removals of COD and suspended solids from this test were 56% and 55% respectively. These removals were significantly higher than those obtained on the same waste (12/21/67) using a 200 mesh screen, i.e., 22% COD removal and 26% suspended solids removal (Table 3). These results indicate that flocculation prior to screening can improve the efficiency of the screens. More detailed data, however, is needed to verify these results.

The results of the laboratory flotation testing is presented in Tables 4 and 5. Table 4 indicates those flotation tests which were run on the raw overflow and Table 5 those tests run on screened overflow. The results shown in Table 4 indicated extremely high removals of COD and suspended solids may be obtained using flotation alone in conjunction with polyelectrolyte addition. The bench scale flotation testing was performed using a standard procedure which is detailed in Appendix II. Removals of 73 to 95% COD and 88 to 97% suspended solids were obtained. In Table 5 data on screening/flotation are presented. All the experiments except 5/20/68 were performed without the addition of flocculating chemicals. Based on the data presented in Table 5, suspended solids removals of 41% to 72%, BOD removals of 40% to 62% and COD removals of 42% to 72% were predicted at the 90% confidence level. By comparing the two experiments on the overflow of 5/20/68,

TABLE 3

SUMMARY OF PRELIMINARY SCREENING DATA

Date of Overflow	Screen Size Mesh	Screen Opening Microns	COD Data Raw mg/1	COD Data Dissolved mg/1	COD Data After Screening mg/1	COD Data Removal %	Suspended Solids Raw mg/1	Suspended Solids After Screening mg/1	Suspended Solids Removal %
11/10/67	50	297	520	110	407	22	418	270	35
11/16/70	50	297	210	---	182	13	---	---	--
12/07/67	50	297	298	50	247	17	232	---	--
12/21/67	50	297	150	30	121	19	242	195	19
01/29/68	50	297	1410	60	922	35	2158	1526	29
04/03/68	50	297	889	95	559	37	808	707	13
04/03/68	50	297	188	---	121	36	194	183	6
04/17/68	50	297	178	---	134	25	228	207	9
04/20/68	50	297	52	---	51	2	70	67	4
04/23/68	50	297	134	---	99	26	137	122	11
04/28/68	50	297	476	---	331	30	461	331	28
04/28/68	50	297	311	---	199	36	248	177	29
05/20/68	50	297	251	---	188	25	194	153	21
01/29/68	100	145	1410	60	899	36	2158	1504	30
12/21/67	120	122	150	30	66	56	242	68	55*
11/10/67	170	87	520	110	340	35	418	210	50
12/07/67	200	74	298	50	233	22	180	150	17
12/21/67	200	74	150	30	117	22	242	178	26
01/29/68	200	74	1410	60	---	--	2158	1368	37
12/07/67	400	37	298	50	200	33	180	100	44
01/29/68	400	37	1410	60	---	--	2158	900	58

* Flocculated with 1 mg/1 Reten A-1 (Hercules, Inc.)

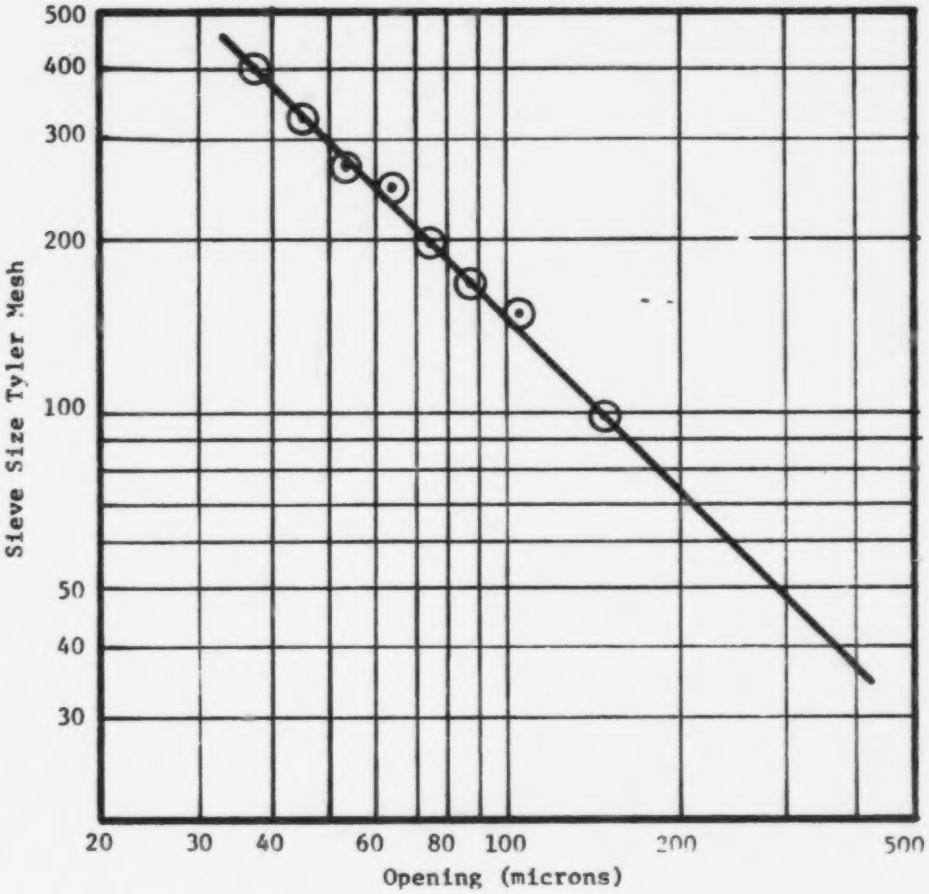


FIGURE 6

COMPARISON TYLER MESH TO SIZE OF OPENING

TABLE 4

SUMMARY PRELIMINARY FLOTATION DATA

Date of Overflow	Pressurized Flow - %	Deten Time Min.	Chemical		COD Data			Suspended Solids		
			Type	Dosage mg/1	Raw mg/1	Effluent mg/1	Removal %	Raw mg/1	Effluent mg/1	Removal %
12/21/67	15	3	Reten A-1(1)	1	150	37	75	242	17	93
12/21/67	8	3	Reten A-1(1)	1	150	40	73	242	28	88
1/29/68	15	5	C-31(2)	10	1410	98	93	2524	75	97

NOTES:
1. Anionic polyelectrolyte-Hercules, Inc.
2. Cationic polyelectrolyte-Dow Chemical

TABLE 5

PRELIMINARY SCREENING/FLOTATION DATA

	4/3/68	4/3/68	4/17/68	4/20/68	4/23/68	4/28/68	4/28/68	5/20/68	5/20/68(1)
SUSPENDED SOLIDS									
Raw mg/1	808	194	228	70	137	461	248	194	194
After Screen mg/1	707	183	207	67	122	331	---	153	153
Screen & flotation mg/1	279	114	73	55	65	65	---	68	36
Overall Removal %	65	41	68	21	53	86	---	65	81
COD									
Raw mg/1	889	188	178	52	134	476	311	251	251
After Screen mg/1	559	121	134	51	99	331	199	188	188
Screen Flotation mg/1	---	---	69	44	64	119	93	114	92
Overall Removal %	---	---	61	15	52	75	70	56	63
BOD									
Raw mg/1	181	36	47	13	26	111	65	54	54
After Screen mg/1	148	33	33	10	20	92	58	49	49
Screen & flotation mg/1	85	12	21	10	12	36	27	39	29
Overall Removal %	53	67	55	23	54	68	58	28	46

(1) 10 mg/1 C-31 added to flotation test - all other runs without chemicals

pressurized flow - 20%; pressure 50 psig; detention time - 5 minutes the effect of chemical flocculant addition may be seen. Addition of 10 mg/l C-31, a cationic polyelectrolyte, caused and increase in suspended solids removal from 61 to 81%, while the BOD and COD removals increased from 56 and 28% to 63 and 46% respectively.

A summary of the preliminary disinfection data collected during the preliminary sampling period is presented in Table 6. Data from ozone and chlorine disinfection are shown. In general, ozone disinfection was not as reliable as chlorine. Ozone demand for these waters was quite high, and since ozone is extremely reactive, a residual ozone concentration could not be obtained except at extremely high dosages (approximately 60 mg/l or greater). Inability to obtain small residual ozone concentrations for a short period of time resulted in the relatively poor E. Coli removals.

Chlorine disinfection on the other hand provided improved disinfection at a dosage of 10 mg/l. Since chlorine is less reactive compared to ozone, some chlorine residual could generally be attained. Disinfection efficiency was better than ozone, but in some tests (April 28, 1968) with high coliform densities, the effluent had significant coliform indicating incomplete disinfection. It appears from this data, as the coliform density increases, higher chlorine dosages will be required.

Conclusions -- Preliminary Investigations

The following conclusions can be made based on the data taken during the preliminary investigations.

1. Combined Sewer overflow from the Hawley Road sewer contains primarily particulate pollution.
2. Chemical oxidation is not technically feasible for combined overflows.
3. Screening/dissolved-air flotation is a relatively effective method for treating combined overflow.
4. Addition of chemical flocculents greatly increases the removal efficiency of dissolved-air flotation.
5. Disinfection with chlorine was found to be more reliable than with ozone.

TABLE 6

PRELIMINARY DISINFECTION DATA

Date	Type	E. Coli. per ml	Ozone Dosage mg/1	E. Coli. in Effluent per ml	Chlorine Dosage mg/1	E. Coli. in Effluent per ml
3 April 1968	FF	1,388	80	21	--	--
3 April 1968	EO	421	59	4	--	--
17 April 1968	EO	1,280	40	74	10	0.5
20 April 1968	EO	1,850	~30	17	10	1
22 April 1968	EO	6,000	<10	3,200	10	2
28 April 1968	FF	32,000	<10	9,500	10	20
28 April 1968	EO	26,000	<10	13,700	10	8

FF - First Flush
EO - Extended Overflow

SECTION VI

DESIGN AND CONSTRUCTION OF DEMONSTRATION FACILITY

The results of the preliminary sampling and laboratory analysis were utilized in the design of a 5 MGD demonstration treatment facility incorporating screening and dissolved-air flotation. This section of the report describes the design criteria utilized, and the features of the demonstration system. A flow sheet for the system is shown in Figure 7 and a photograph of the overall system is shown in Figure 8.

Design of Screen

The design of the demonstration unit involved three basic areas, *i.e.*, screen design, flotation tank design and integration of all system components. Based on removal efficiencies obtained in the preliminary sampling phase, a screen mesh of 50 (297μ openings) was selected. This mesh gave fairly good removal of pollutants (20-30%) and allowed high flow rates (50 gpm/sq ft) at reasonable headlosses. The screen is an open ended drum into which the raw waste flows after passing a 1/2" bar rack. The purpose of the bar rack is to remove large objects which may clog the screened solids removal system or damage the screen material, which is 306 stainless steel. The water passes through the screen media and into a screened water chamber directly below the drum. The drum rotates and carries the removed solids to the spray water cleaning system where they are flushed from the screen. Those solids which will not adhere to the screen media are picked up by 4 angle iron sections which act similar to a roto-dip feeder and are thus removed from the flow. Screened water is used for backflushing the screen. The drum rotation and spray water cleaning are controlled by liquid level switches located in the screen chamber. The switches are set to actuate at a headloss of 6" of water through the screen. The solids which are flushed from the screen along with the spray water are collected in a hopper inside the drum. This hopper is connected to a drain pipe which forms the main axis of the drum. The slurry is then routed to waste. Detailed photographs of the screening system are shown in Figures 9 and 10. These photographs show the screen, internal hopper. float switches, and drive system. The basic screen is fabricated from mild carbon steel. The screen backing material (Figure 9) is a perforated metal plate which has proved to be adequate support for the screen. Hole size for the backing material is 3/4" x 3/4" on 7/8" centers. This provides 73% open area and only a minimum of flow restriction. Rotation speed is controlled by a variable speed drive, which is positioned manually. Rotation speed range is 0.5 - 5 rpm. The screened water is sealed from the raw waste by compressing a tube between the end of the drum and a stationary ring. The sealing arrangement allows operation at a maximum headloss of about 12". Any excess headloss will allow some unscreened water to enter the screened water chamber and a possible raw waste overflow from the bar screen chamber. During some runs with extremely heavy solids loads, the headloss capacity of the screen has been exceeded. These

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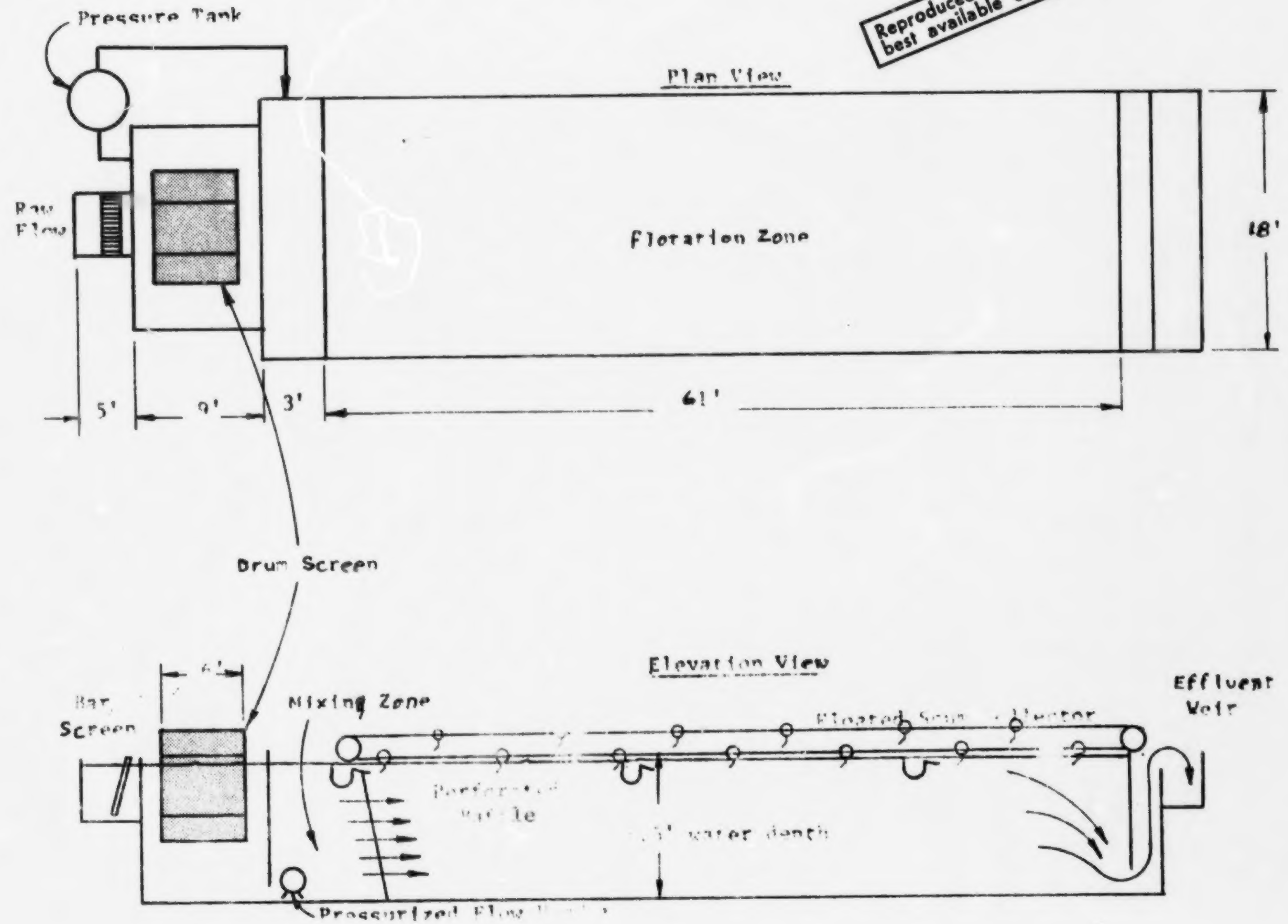


FIGURE 7

DETAILS OF DEMONSTRATION SYSTEM

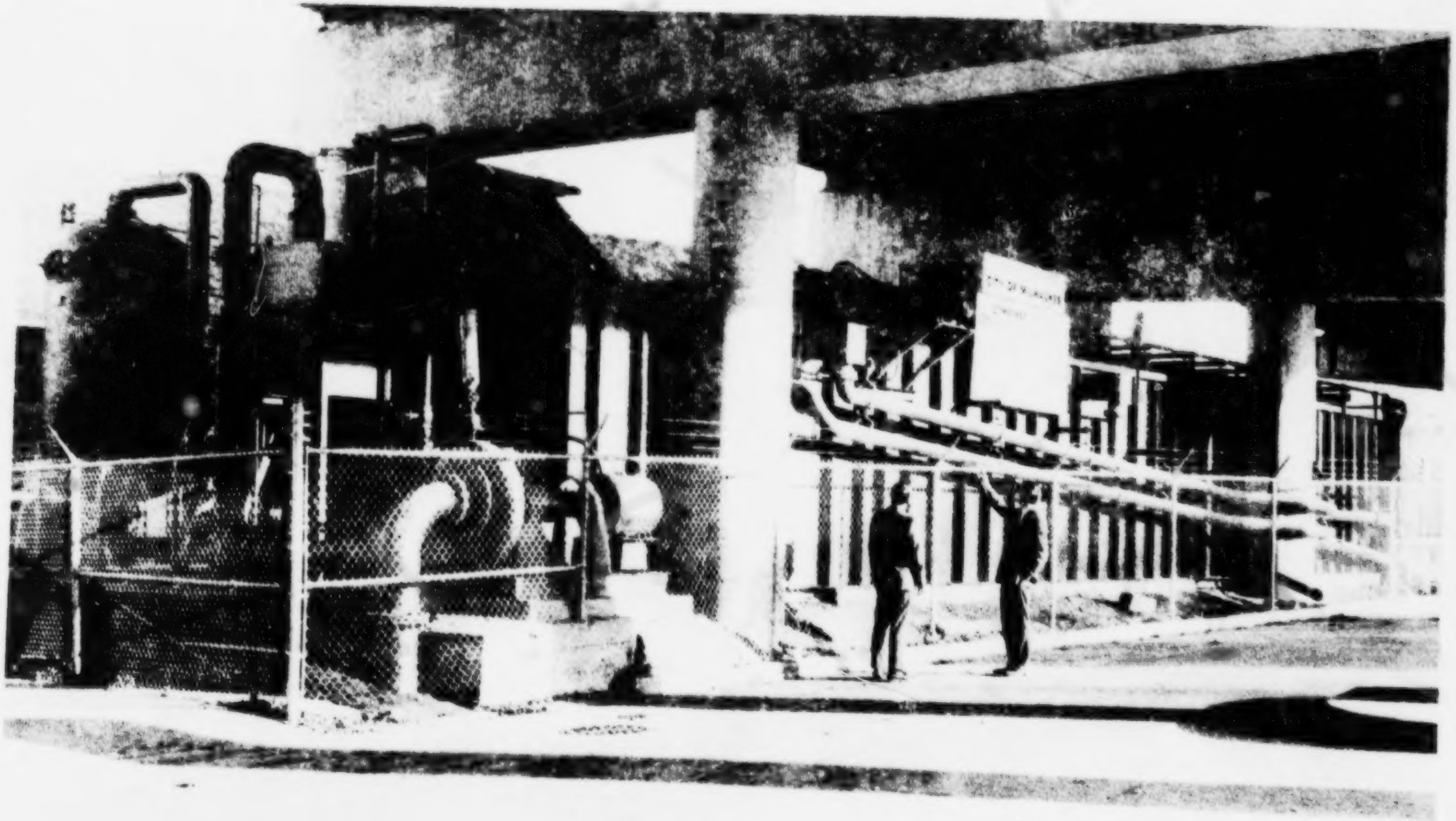


FIGURE 8

DEMONSTRATION SYSTEM

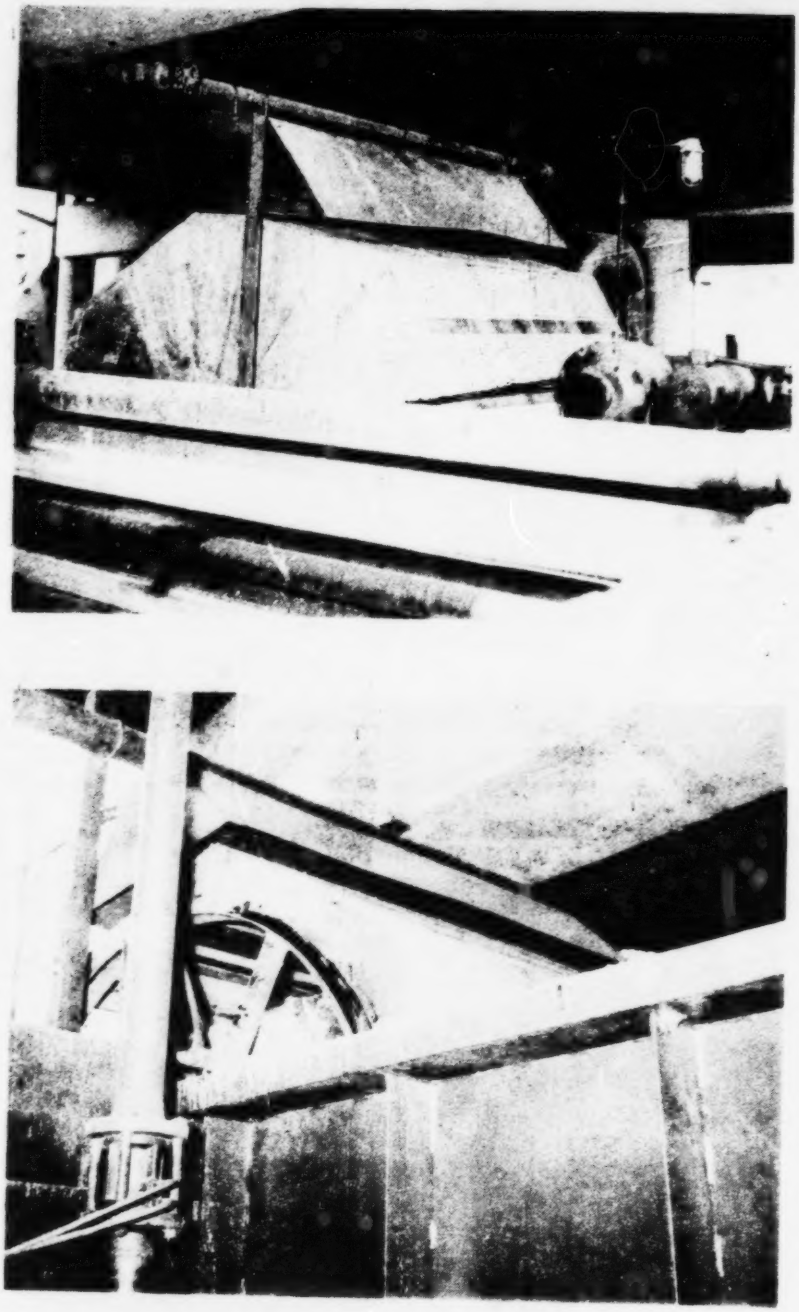


FIGURE 9

SCREENING SYSTEM

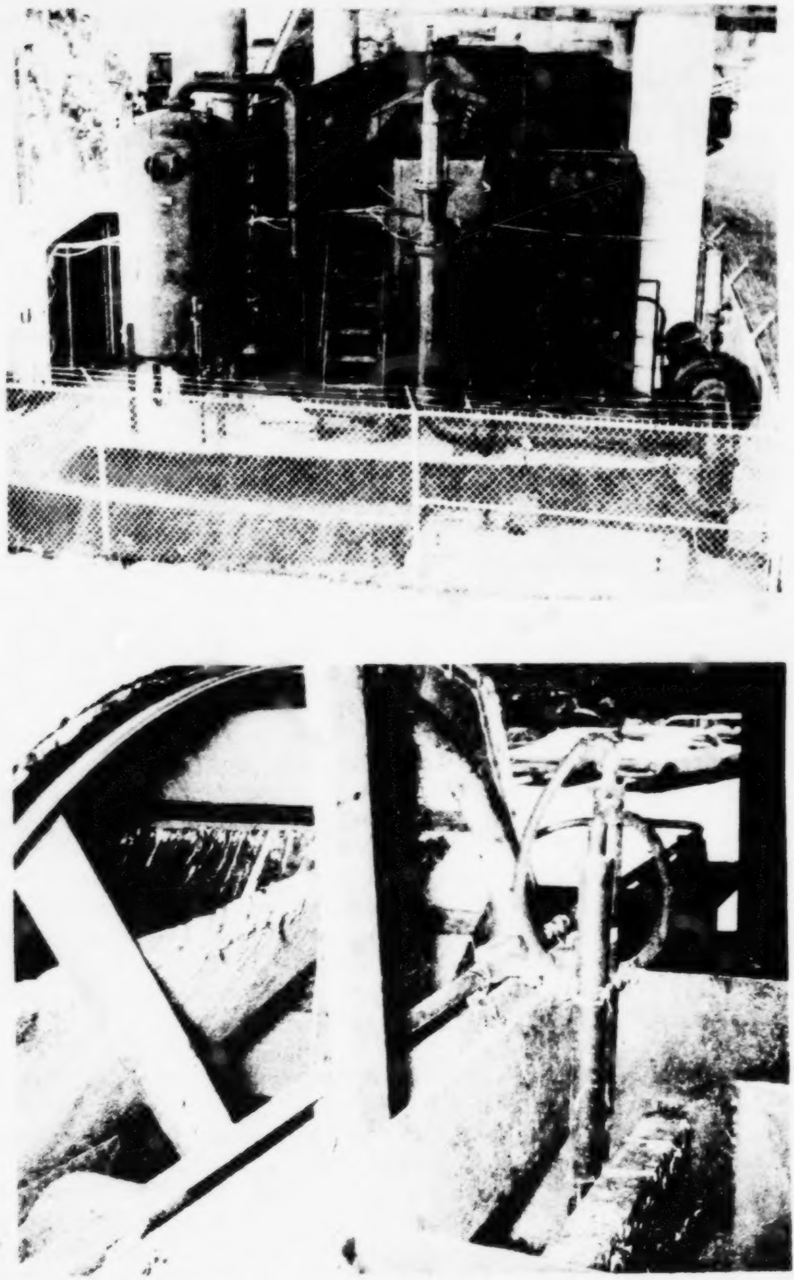


FIGURE 10

SCREENING SYSTEM

instances will be discussed in a later section of this report.

The drum screen as installed is an 8 sided drum with an effective diameter of 7½ feet. The length of the drum is 6 feet. The 8 panels have dimensions of 3' wide by 6' long. Total screen area is 144 sq ft. The wetted screen area was a minimum of 72 sq ft and a maximum of 90 sq ft depending upon the head loss across the screen.

Flotation System Design

The dissolved air flotation basin design was based on the criteria utilized by the American Petroleum Institute (API)(64). The major design parameters are overflow rate, detention time, horizontal velocity and depth to width ratio. The design procedures were modified slightly to allow the high flexibility required in a project of this nature.

The basic principle of dissolved air flotation (DAF) is to produce extremely small air bubbles ($<100\mu$) which can be attached to the particulate matter in a wastewater and cause flotation and removal of the particulate matter. To provide these fine air bubbles a liquid stream is mixed with air under pressure. The pressure is then released through a weir type diaphragm valve to form the bubbles. The bubble laden stream is then mixed with the remaining wastewater to be clarified in a contained mixing zone within the DAF tank. This mixing zone has a detention time of approximately 60 seconds. The bubbles attach to the solids in this mixing zone. The bubble formation system is termed the pressurized flow system. Generally, the source of pressurized flow is the process effluent or another relatively solids free stream. In this project the screened water was used as the source of pressurized flow. The advantages to this approach eliminate an increase in hydraulic loading on the DAF tank and provides a simplified plumbing system. Because of the possible presence of a significant amount of solids in the pressurized flow stream, a pressure tank without packing material was utilized. This avoided the potential clogging problems associated with a pressure tank packed with some form of tower packing to increase the air water interface. The type of pressurization utilized in this project has been termed sidestream-pressurization.

The following discussion will delineate the process steps occurring in the flotation tank. Screened water is pumped into the pressure tank. Air is mixed with the water at the inlet to the pressure tank. Water level in the tank is controlled by a liquid level float switch. A slight excess of air is always added. If the water level becomes too low the air stream is vented to atmosphere to allow the water level to rise. Hence, the water level in the pressure tank is positively controlled. There is a deflector baffle in the tank which spreads the incoming water and promotes a large air water interface for good air solution. The pressure in the tank is controlled by an air operated weir valve; this valve provides the proper back pressure as well as the required shearing action to form the small bubbles. This valve is controlled by a pneumatic controller. Once the pressure has been released and the bubbles formed, the bubble laden stream is mixed with the remainder of the screened water flow. A 60 second mixing chamber is provided in the tank prior to entering the flotation zone. This allows time for bubble/solid attachment. When flocculating chemicals are utilized they may be added either to the raw flow or to the pressurized flow after the pressure has been reduced. Generally polyelectrolyte type flocculants, which require little or no flocculation period are added in the pressurized flow stream. Once the pressurized stream and remaining raw waste have been mixed, they enter the flotation zone of the tank where separation occurs. A skimming system skims the floated scum into three separate scum troughs, which convey the scum to ultimate disposal. Generally the solids concentration of the scum is 1-2% solids on a dry weight basis. This concentration flows easily by gravity and does not require screw conveying. Figures 11 and 12 show various details of the flotation system.

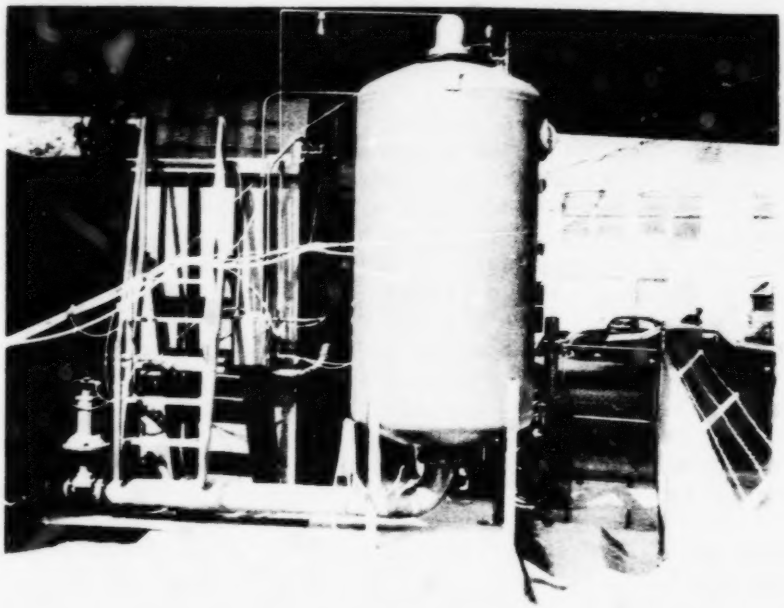
The design of the DAF system was such that a wide range of selected variables could be evaluated in order to be able to recommend design procedures specific to combined sewer overflow. The following range of variables is possible with the demonstration system.

Flow rate	1500 - 4400 gpm
Surface loading	2 - 10 gpm/sq ft
Horizontal velocity	1.30 - 3.75 ft/min
Pressurized flow rate	300 - 1100 gpm
Operating pressure	40 - 70 psig
Detention time	7 - 44 minutes

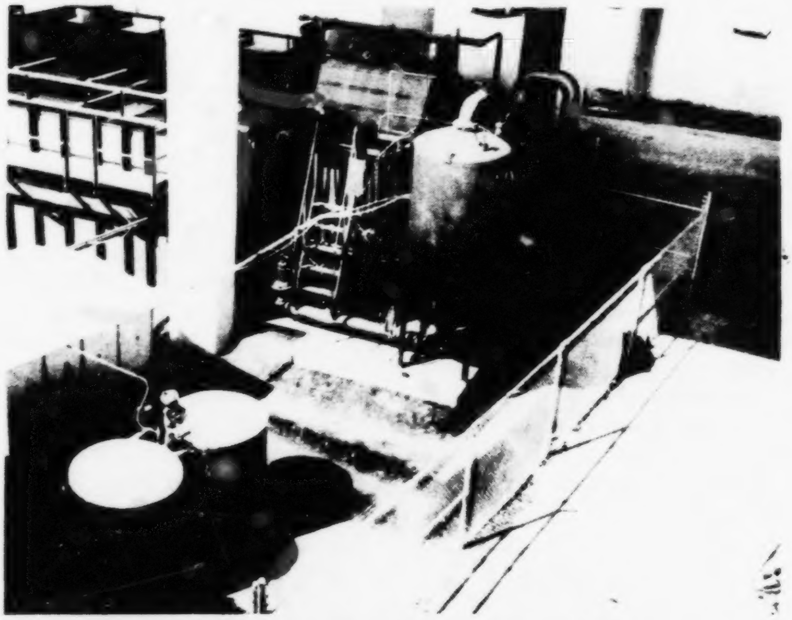
In addition to the above flexibility, the flotation tank can be divided down the center. This in effect forms two separate flotation tanks. The lengths of these tanks can be controlled by appropriate baffels. This then allows the evaluation of two separate overflow rates on the same storm, thus allowing direct comparison of the overflow rate variable and elimination of all other possible interacting variables. The tank was divided as described above at the start of the 1970 storm period (April, 1970). The final as-built dimensions of the flotation tank were 18 feet wide, 8½ feet water depth, and a 65' long flotation zone.

Design of Supporting Systems

The appurtenant equipment and tasks necessary to provide a functional demonstration system included site preparation, construction of a manhole and pumping sump, selection of the necessary flow metering equipment, and design of suitable electrical and pneumatic control systems.



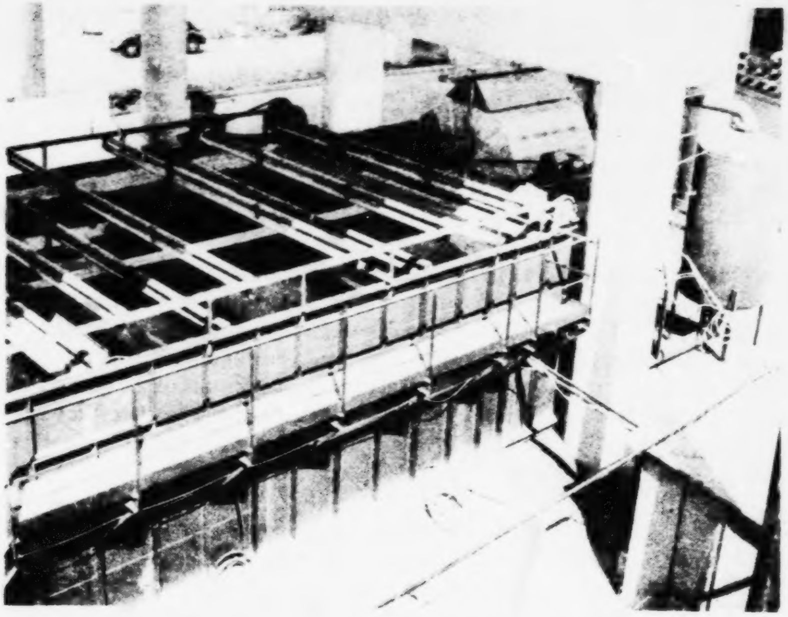
PRESSURE TANK, CONTROL SHACK, AND SCREEN SYSTEM



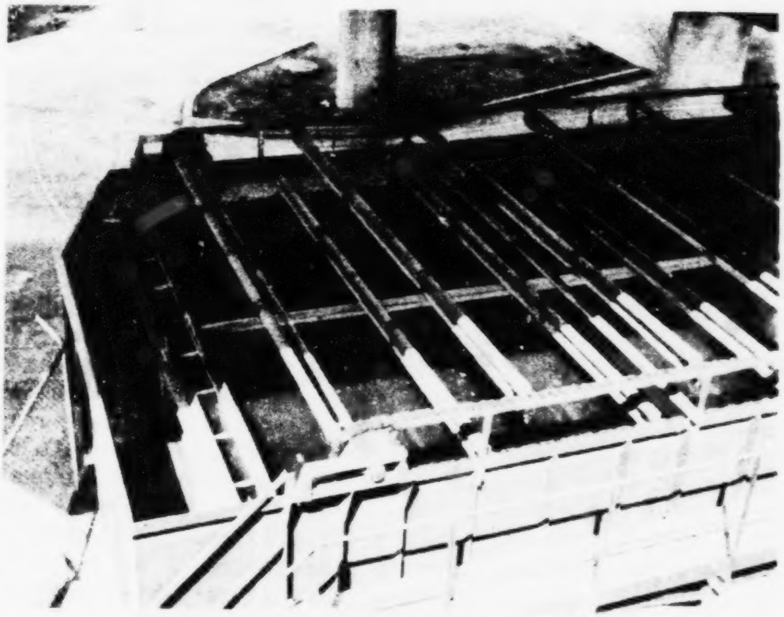
PRESSURE TANK AND PRESSURE REDUCTION VALVE

FIGURE 11

FLOTATION SYSTEM (See also Figure 7)



HEAD END OF FLOTATION TANK



OVERHEAD COLLECTOR AND EFFLUENT STRUCTURE

FIGURE 12

FLOTATION TANK

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As may be seen in Figure 8, the system was located under an existing highway bridge which provided overhead protection. Site preparation involved laying a concrete slab to provide support for the system tankage. A manhole was constructed as well as a pumping sump, directly in the combined sewer. A dam was also provided in the sewer to allow impoundment of a limited quantity of overflow for subsequent treatment. This dam was designed to tip under the hydraulic pressure of the overflow to prevent surcharging the sewer. A photograph of the outfall is shown in Figure 13.

Flow metering equipment is provided to measure the influent flow rate, the volume of screen backwash water and the volume of floated scum. All other process flows could be obtained by adding or subtracting the proper measured values. The raw flow and screen backwash are measured via venturi meters connected to differential pressure gauges which both record and totalize the flows. The floated scum is measured with an open channel float type meter, which records and totalizes the flow of floated scum.

The electrical control panel provides all necessary controls for 100% automatic operation with manual overrides on all systems. Operational modes will be discussed in a later section of this report. A Merchants Police alarm is connected to the system so that personnel will be alerted when the system goes into operation. The system is always (24 hours per day, 7 days per week) ready to operate, and hence, the maximum possible number of overflows can be monitored. Photographs of the control shack are shown in figure 14. Demonstration system costs are presented in Appendix III.

Operation Methods and Test Plan

Methods and Operational Procedures

The demonstration system previously described was put into operation in May of 1969. Data reported herein represent data taken during the period May, 1969 through November, 1970. During this period 55 overflows were treated with the demonstration system. The system is put into operation automatically when a float switch in the sewer senses an overflow. The pressurized flow system is immediately put into operation and the raw feed pump begins to prime. All runs were started with the tank approximately 80% full of water. The water in the tank was that left from the previous run. The raw feed pump generally primed in about 12-15 minutes. When primed the raw pump is activated, and the flow meters, chemical feeder (if utilized), skimmers and all other auxiliary equipment are put into operation. At the end of the run the system shuts down automatically. All variables are then selected for the next run and the controls positioned. Variables associated with tank operation include pressurized flow rate, operating pressure, scum removal cycle, raw flow rate, and chemical dosage.

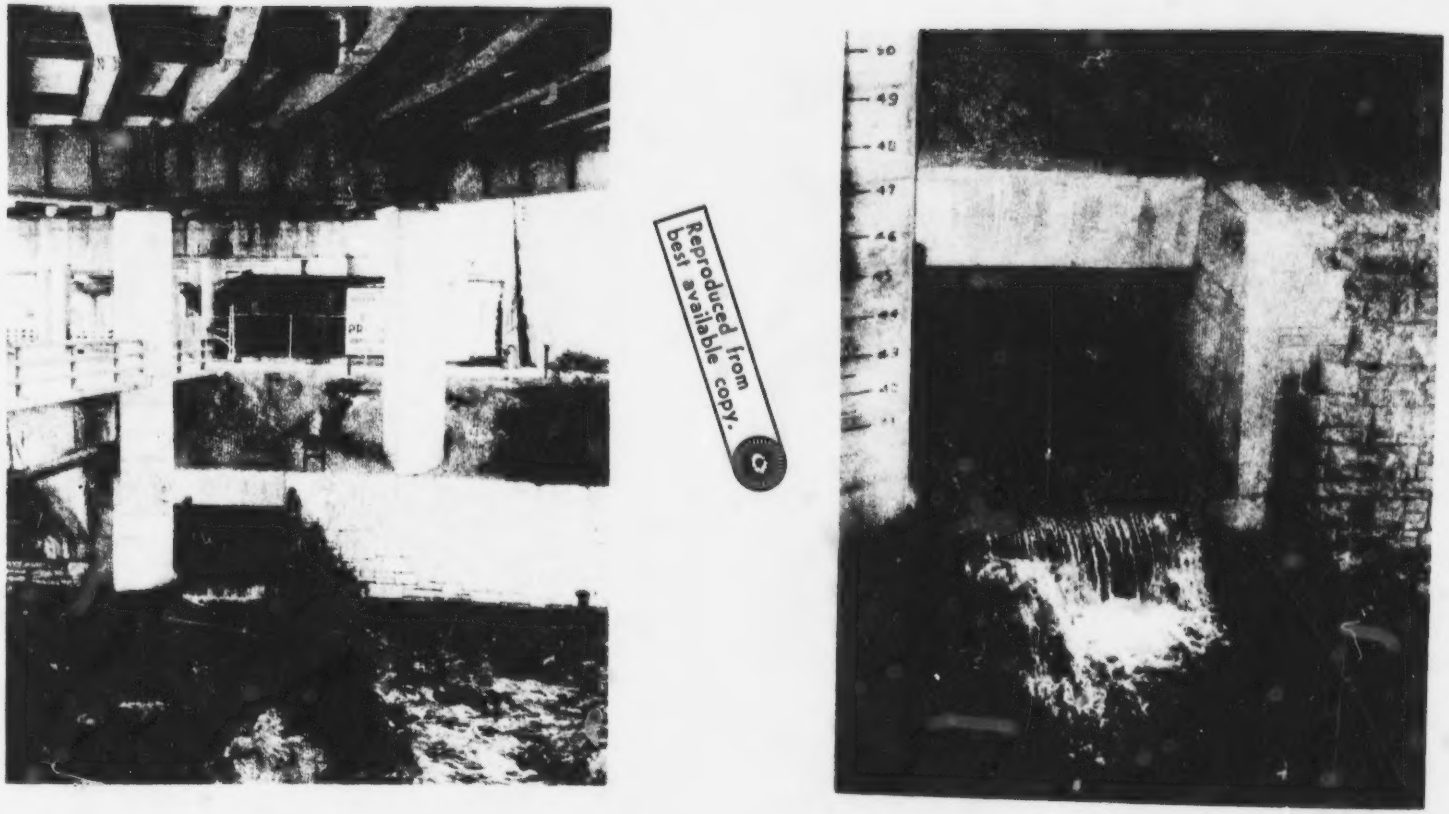

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FIGURE 13

HAWLEY ROAD OUTFALL

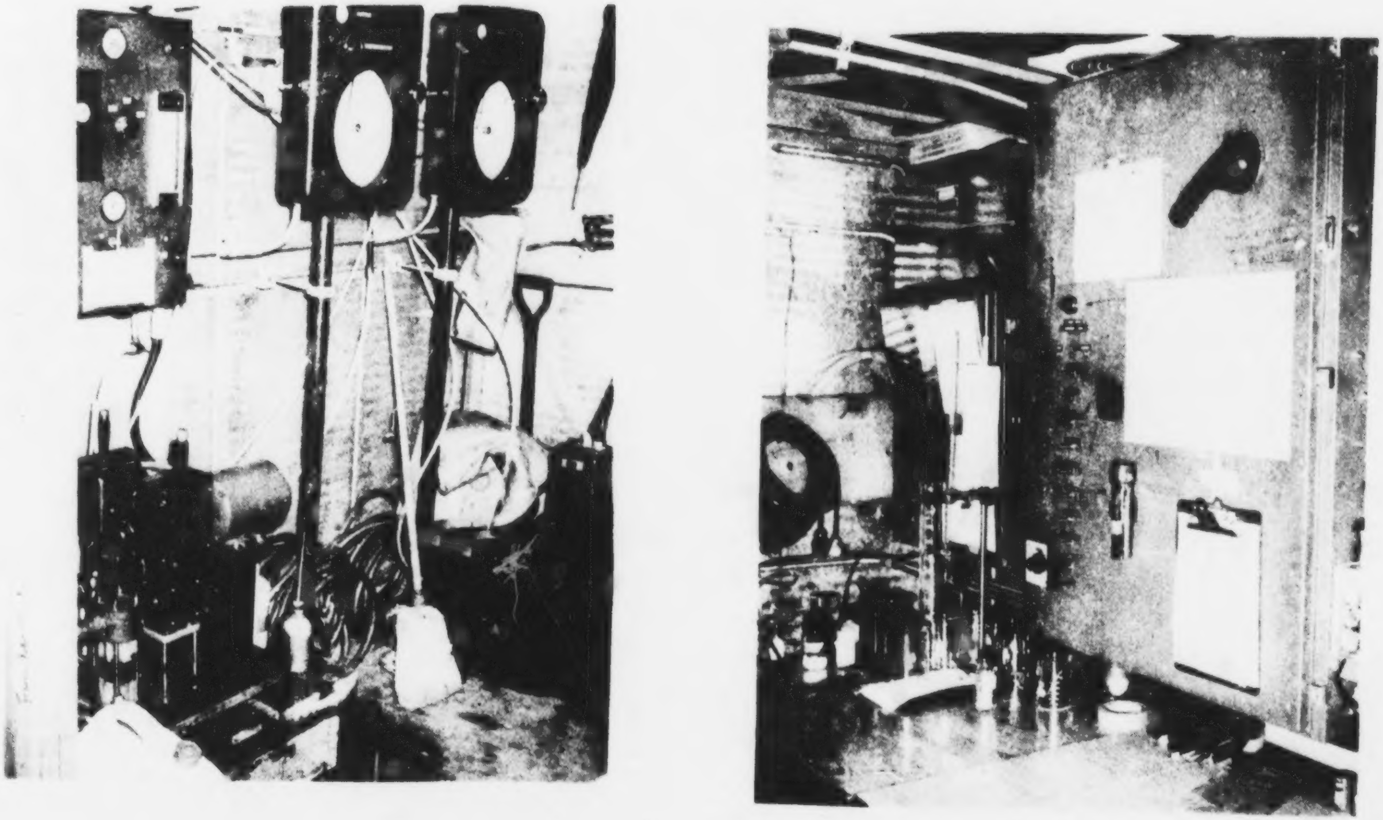

FIGURE 14

INTERIOR VIEWS OF THE CONTROL SHACK

Sampling Procedures

Sampling began when the raw pump primed. Raw waste and screened water sample collection was started immediately. Effluent sample collection was delayed for 15 minutes to allow purging of the water in the tank from the previous run. This procedure insured collection of representative effluent samples. Screen backwash and floated scum samples were taken during screen backwash and scum removal periods.

Sampling during 1969 runs was manual. Equal volumes of raw waste were taken every ten minutes. Screened water and effluent samples were taken continuously at a rate producing about 2.5 gallons of sample per hour. Screen backwash and floated scum were composited in equal volumes.

An automatic sampling system was put into operation for all 1970 runs. The system consists of two timers connected through the proper valving, to automatically composite the raw waste, screen water and effluent samples. The first timer controls the sample taking frequency (0-30 minutes). The second timer controls the duration of sampling time (0-60 seconds). The sampling valves are air operated weir valves. The automatic system has proved much superior to the manual methods, since the chance for human error has been eliminated and the operator is free to monitor the remainder of the system. In general samples were composited every 5 minutes with the automatic system. Floated scum and screen backwash sample taking was not automated due to the problems associated with intermittent flows and heavy solids concentrations in these process streams. The samples were refrigerated immediately after the run. Analysis were then started within 0-8 hours. Sample analysis procedures are discussed in Appendix II.

Test Plan

Variables associated with the operation of the demonstration system include hydraulic overflow rate (gpm/sq ft of tank area), pressurized flow rate, operating pressure, addition of chemical flocculants, and the floated scum removal cycle. During the first few runs it was determined that values for operating pressure and the removal cycle for floated scum were not critical. A test plan was then initiated with pressurized flow rate, hydraulic overflow rate, and addition of chemical flocculants as the three variables. Eight possible combinations of these variables are shown in Table 7. It was planned to obtain 7 to 8 runs for each variable combination requiring 56 to 64 separate runs. The demonstration system was operated on 55 combined sewer overflows. All variable combinations of Table 7 were fully evaluated except Numbers 3 and 4. These combinations were abandoned based on the results obtained from combinations 1 and 2 which indicated higher overflow rates without chemicals was not feasible.

TABLE 7

VARIABLE COMBINATIONS UTILIZED FOR TESTING

Combination	Pressurized Flow as % of Total Flow	Overflow Rate gpm/sq ft	Chemical Flocculants
1	14-20	2.5	No
2	21-30	2.5	No
3	14-20	3.8	No
4	21-30	3.8	No
5	14-20	2.5	Yes
6	21-30	2.5	Yes
7	14-20	3.8	Yes
8	21-30	3.8	Yes

SECTION VII

OPERATING RESULTS AND DISCUSSION

Characterization of Raw Waste

As was expected, the quality of the combined sewer overflow from the Hawley sewer varied widely. In 12 of the 55 overflows reported herein, extremely high pollutional values were observed. These first flushes persisted for 20 to 70 minutes. A summary of the first flush data may be seen in Table 8. The range of the data presented is at the 95% confidence level. First flush occurrence appears to be associated with the length of time between overflows and the intensity of the overflow. The suspended solids concentration for those overflows occurring within 4 days of a previous overflow were calculated to be 151± 29 mg/l while the suspended solids for those overflows occurring at an interval larger than 4 days were found to be 349 ± 80 mg/l. Comparison of the COD data at intervals shorter than 4 days and longer than 4 days produced CODs of 144 ± 21 and 394 ± 72 mg/l respectively. These values are at the 95% confidence level. All the data was tabulated as a function of the interval between overflows. (Table IV-10, Appendix IV-11.) This data indicates that essentially all overflows which exhibited the first flush phenomenon occurred at intervals of 4 days or longer between overflows. However, all overflows occurring at an interval of greater than 4 days did not exhibit the first flush phenomenon. Table 9 illustrates this point. All overflows which occurred at intervals of greater than 4 days are presented in Table 9 along with data on rainfall intensity and total rainfall. A total of 23 overflows occurred at intervals of greater than 4 days between overflows. Of these 23 overflows only 12 exhibited the first flush phenomenon. It may be concluded from this data that the length of time between overflows is related to the occurrence of first flushes and there are obviously other variables which strongly influence this phenomenon. These variables could include: the dry weather flow variation, the intensity of rainfall and runoff and the sewer system interceptor capacity. Furthermore the data presented herein may be biased by the 12-15 minutes which was required to prime the raw pump for the treatment system in that extremely short first flushes would not have been detected.

A satisfactory method for measuring the rate of flow in the sewer has not been developed. Development of a method has been complicated due to the presence of the dam, which was placed in the sewer to impound the flow. This dam backs water up past one of the two interceptor devices feeding the Hawley Road sewer. This makes single point gauging impossible. Another complication which arises is the fact that the dam is not stationary.As the flow increases the dam tips to prevent sewer surcharge. This causes large fluctuations in water level. Down stream of the dam the sewer has a dog leg and extreme turbulence has been observed at the mouth of the sewer. Because of the above discussed difficulties reliable data on the total flow rate in the sewer was not obtained. However, lack of total flow data does not in any way influence the operation of the demonstration system.

TABLE 8

SUMMARY FIRST FLUSH DATA

Analysis	Concentration mg/1
COD	581 ± 92
BOD	186 ± 40
Total Solids	861 ± 117
Total Volatile Solids	489 ± 83
Suspended Solids	522 ± 150
Volatile Suspended Solids	308 ± 83
Total Nitrogen	17.6 ± 3.1
Ortho Phosphate	2.7 ± 1.0
pH	7.0 ± 0.1
Coliform Density	142 ± 108 x 10^3 per ml

Data Represents 12 Overflows
95% Confidence Level Range

TABLE 9

SUMMARY OF DATA

AT INTERVALS OF GREATER THAN 4 DAYS BETWEEN OVERFLOWS

Run No.	Days Between Overflows	First Flush Occurrence	Average Rainfall Intensity in/hr	Total Rainfall (inches)
691	18	Yes	0.13	0.42
695	15	Yes	0.32	0.25
6911	8	Yes	0.40	0.50
6912	5	Yes	0.70	1.00
6914	6	Yes	0.35	0.17
6916	12	Yes	0.40	0.10
6919	24	Yes	1.2	0.10
702	26	Yes	0.17	0.3
708	9	Yes	0.84	0.14
7011	14	Yes	0.17	0.25
7013	12	Yes	0.80	0.20
7022	19	Yes	0.10	0.15
6918	11	No	1.6	0.45
6922	17	No	0.3	0.10
6923	6	No	0.5	0.5
6925	11	No	---	0.17
6928	19	No	0.1	0.12
707	7	No	0.4	0.30
7010	11	No	0.11	0.33
7014	6	No	0.18	0.22
7017	8	No	0.36	0.30
7019	18	No	0.15	0.23
7020	15	No	0.24	0.25

Data on rainfall was obtained routinely and runoff rate estimates were made using the rational method. These runoff rates were then plotted against the suspended solids in the overflow. A result of this plot is presented in Figure 15. It appears from Figure 15 that there was no relationship between flow rate in the sewer and the suspended solids in the overflow. This is as expected, since once the sewer system and drainage basin has been thoroughly flushes, the rate of low flow should not have a significant effect on overflow quality. However, since actual sewer flow rates were not obtained, a positive conclusion cannot be made.

After the flushes had passed (if they were present), the characteristics of the overflow became remarkably stable, considering the large variations present in the first flushes. The end of the first flushes was determined by visual observation of the raw combined overflow. The screen backwash ran continuously during periods when high suspended solids were present in the raw feed. Thus when the screen backwash pump began cycling the end of the first flushes was indicated. The exact length of the first flushes was not recorded. A summary of all data other than first flushes (termed extended overflows) is presented in Table 10. The data of Table 10 show a relatively small range of values at the 95% confidence level. The data compared well to data from other research on combined sewer overflow (6)(11)(12)(13)(14)(65)(66). The pollutant levels in the extended overflow are about what is expected from a very weak domestic sewage. One major difference is the BOD value of 49 mg/l. This is quite low compared to the COD value of 161 mg/l. Generally, the BOD:COD ratio of domestic sewage is in the range of 0.6 (67). The dissolved solids in combined overflow is also quite low. Total dissolved solids (TDS) based on the data of Table 10, is 212 mg/l. Milwaukee tap water is about 160 mg/l. This represents only about 42 mg/l TDS increase as compared to ~700 mg/l usually added when water is used and discarded as domestic sewage (8).

Of particular interest ot this project is the amount of organic material present in the dissolved state. Table 11 presents data on this relationship. The amount of dissolved COD ranges from 30 to 38% of the total COD. The dissolved TOC ranges from 26 to 42% of the total TOC. These values are somewhat higher than those obtained in the preliminary sampling phase of 4-25%. The cause of this difference is not known. The dissolved portion of the overflow will not be removed in the screening/flotation system, unless it is chemically precipitated. Hence, lower removal efficiencies will be expected as the dissolved fraction of the waste increases.

A discussion of the results of the operation of the demonstration unit follows. A discussion is divided into three phases: screening operation, flotation tank operation and disinfection. Because of the wide range of pollutional values obtained in the raw waste, the removals through various processing steps are presented as percentage removals. Absolute effuent water quality data may be seen in Appendix IV, Tables IV-6 and IV-7. Absolute effluent quality can also be estimated by applying the observed percentage removal figures (Table 12 and 13) to the raw overflow water quality presented in Tables 9 and 10.

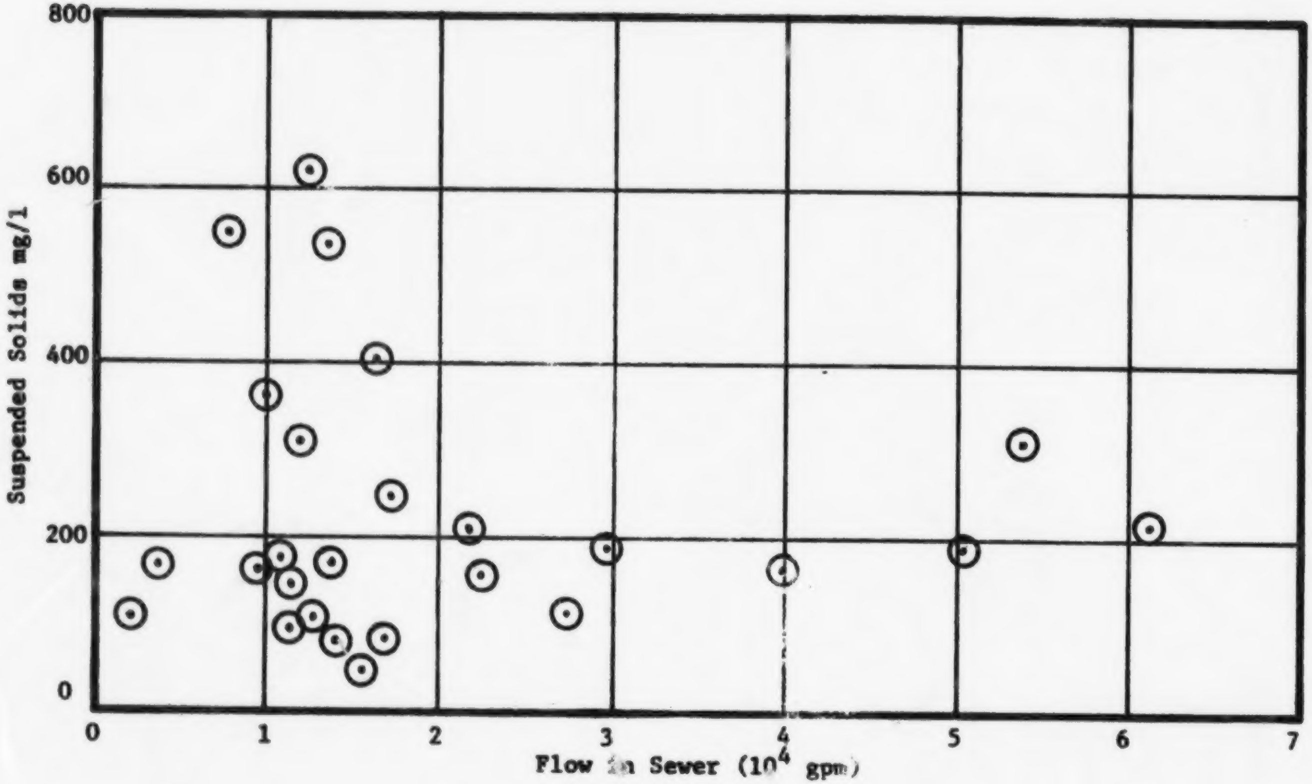

FIGURE 15

RELATIONSHIP BETWEEN FLOW IN SEWER AND SUSPENDED SOLIDS

TABLE 10

SUMMARY OF EXTENDED OVERFLOW DATA

Analysis	Concentration mg/1
COD	161 ± 19
BOD	49 ± 10
Total Solids	378 ± 46
Total Volatile Solids	185 ± 23
Suspended Solids	166 ± 26
Volatile Suspended Solids	90 ± 14
Total Nitrogen	5.5 ± 0.8
pH	7.2 ± 0.1
Coliform Density per ml	$62.5 \pm 27 \times 10^3$

Data represents 44 overflows
at 95% confidence level range

TABLE 11

PARTICULATE & DISSOLVED RELATIONSHIPS

Relationship	No. of Samples	95% Confidence Level
Dissolved COD/Total COD	34	0.34 ± 0.04
Dissolved TOC/Total TOC	13	0.34 ± 0.08
Dissolved TOC/Dissolved COD	14	0.36 ± 0.02
Total TOC/Total COD	17	0.33 ± 0.05

Range at the 95% confidence level

Results of the Screening Operation

Table 12 presents a summary of the data on removal of pollutants by screening alone. A listing of all data collected is presented in Appendix IV. Generally, the percent removals during the first flush were in the range of 30 to 40%. The confidence ban is wide due to the relatively small number of first flush overflow occurrences. During the first flushes a mat of solids sometimes covered the entire screen. This mat acted as a fine filter medium and undoubtedly, increased the removal efficiencies as compared to a clean screen. Unfortunately the headloss increased greatly as the mat formed and in runs 695, 6911, 6912, 6914, 6916 and 702, the headloss capacity of the screen was exceeded for brief periods of time. Examination of these runs indicated that the screen was removing about 200 mg/l of suspended solids which was equivalent to 4.9 pounds of dry solids per minute. Based on screen rotation speed and area, this amounts to a solids loading of approximately 1.2 pounds of dry solids removed per 100 sq ft of screen area. This number represents the critical level of solids loading at which a headloss of 14 inches will be exceeded.

This value of solids loading is specific for a 14 inch headloss across the screen. The loading could possibly be increased by increasing the allowable headloss differential. This could, however, cause a decrease in removal efficiencies by forcing more solids through the screen and/or cause a break up of solids which could affect the efficiency of the flotation process.

The hydraulic flow rate was in the range of 40-45 gpm/sq ft. This rate depending upon solids loading, could probably be increased.

The design of the screening system should be based on both hydraulic flow rate and solids loading rate. Either of these variables could control screen design depending upon waste characteristics. It is obvious that proper screen design requires additional study. All facets of screening combined sewer overflows will be studied in Phase VI of this project which is scheduled for completion in March of 1972.

During the extended overflows and after the first flushes had passed, removal efficiencies dropped to the 20 to 30 percent levels (Table 12). The probable cause of this decrease is that essentially no solids mat formed during extended overflows, and hence, no added filtering action was obtained.

The screen was backwashed with water from the screened water chamber via a pressurizing pump. Spray nozzles with 1/4" diameter orifices were used to effectively distribute the water over the screen media. Washing was performed from outside the screen drum. Some problems were encountered with nozzle plugging due to an inefficient seal on the drum screen. The seal has been improved significantly, but it is still recommended that an inline small hydraulic cyclone be utilized on future designs to eliminate any operational problems. The volume of screen water required

TABLE 12

POLLUTANT REMOVALS BY SCREENING

Pollutant	Removal During First Flushes %[1]	Removal During Extended Overflows %[2]
COD	39 ± 15	26 ± 5
BOD	33 ± 17	27 ± 5
Suspended Solids	36 ± 16	27 ± 5
Volatile Suspended Solids	37 ± 18	34 ± 5

1. Represents 8 overflows (see page 53 to 56 for discussion)
2. Represents 46 overflows

Data at 95% confidence level

is approximately 100 gallons per minute. Continuous washing of the screen would therefore require about 3% of the raw waste flow. Actual spray wash requirements were in the range of 0.7 to 1.0 percent of the raw flow, since the screen wash did not run continuously during operation. Screen wash quality was in the range of 500 to 3000 mg/l suspended solids. (Appendix IV - Table IV-9.) Good media cleaning was always obtained and no permanent media blinding was experienced. In general operation of the screen was very satisfactory. Tentative design criteria for the screen system will be presented in a later section of this report and final specifications at the completion of the project.

Operation of the Flotation System

Overall contaminant removals using the screening and flotation systems are presented in Table 13. A listing of all data is presented in Appendix IV. During the first flushes removals of BOD and COD were in the range of 55 to 65 percent, while suspended solids removals were 70 to 75 percent. Removals of nitrogen were significantly lower at about 46 percent. In those first flush runs where flocculating chemicals were added the optimum chemical dosage was generally not obtained, since the pollutant levels were significantly higher compared to extended overflows. This difficulty illustrates the need for some type of control system which would automatically adjust the chemical dosage depending upon the raw feed water quality.

Removals during extended overflows were generally lower than during first flushes. Removals also varied widely depending upon whether flocculating chemicals were utilized. Table 13 presents various pollutant removal efficiencies for three different categories, i.e., without chemical flocculant addition, with polyelectrolyte (Dow C-31) and clay addition, and with polyelectrolyte (Dow C-31) and ferric chloride addition.

Figure 16 presents a probability plot of suspended solids removal for the 1969 data. The slope of the probability line shows a distinct change at about 50% removal. This change in slope indicates some operating variable was affecting the removal rates and caused a change in the normal distribution of the data. This change was chemical flocculant addition. Examination of the data more closely indicated 10 of 14 runs without chemical addition were below 52% suspended solids removal, while 10 of 13 runs with chemical addition were above 52% removal. Removals of volatile suspended solids show trends similar to suspended solids removals.

The effect of optimizing the chemical treatment scheme is clearly illustrated in Table 13. Suspended solids and volatile suspended solids removals increased significantly by varying the chemical addition from polyelectrolyte and clay (1969 data) to polyelectrolyte and ferric chloride (1970 data). The chemical dosage required to provide consistent removals was found to be 20 mg/l ferric chloride and 4 mg/l polyelectrolyte. The ferric chloride is added to the raw waste water flow prior to screening

TABLE 13

POLLUTANT REMOVALS BY SCREENING/FLOTATION

Pollutant	During First Flushes %(1)	Removal During Extended Overflows - %(2)		
		Without Chemical Flocculants (1969-1970 Data)	With Chemical Flocculants (1969 Data)(3)	With Chemical Flocculants (1970 Data)(4)
COD	64 ± 6	41 ± 8	40 ± 14	57 ± 11
BOD	55 ± 8	35 ± 8	46 ± 17	60 ± 11
Suspended Solids	72 ± 6	43 ± 7	59 ± 11	71 ± 9
Volatile Suspended Solids	75 ± 6	48 ± 11	58 ± 10	71 ± 9
Total Nitrogen	46 ± 7	29 ± 14	19 ± 11	24 ± 9

All Data at 95% confidence level
Overflow Rate ~2.5 gpm/sq ft

(1) Represents 12 overflows
(2) Represents 38 overflows
(3) 2.5 - 3.5 mg/l C31, 6 mg/l Clay
(4) 3-6 mg/l C31, 20-25 mg/l $FeCl_3$

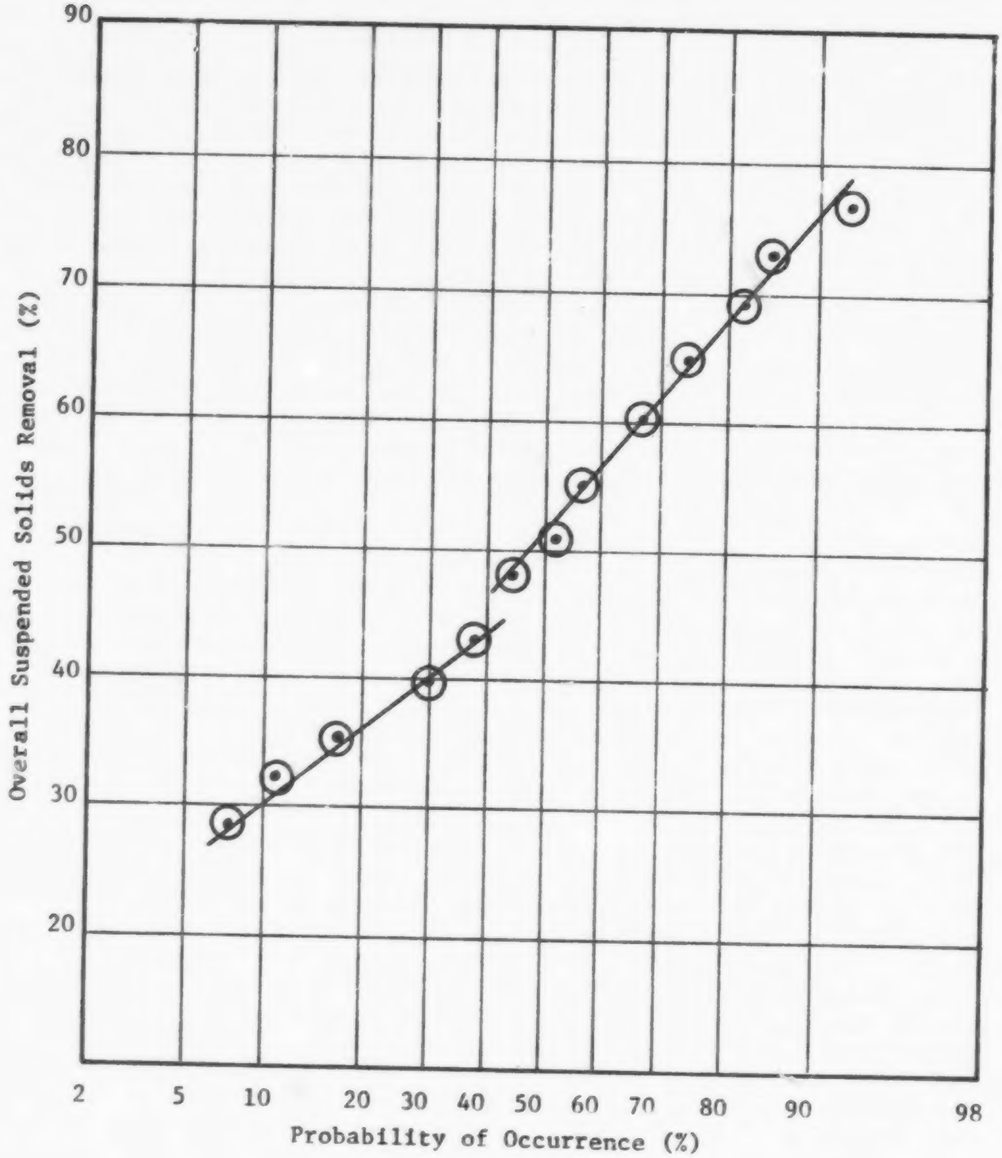

FIGURE 16

SUSPENDED SOLIDS REMOVAL-SCREENING/FLOTATION and the polyelectrolyte is added to the pressurized flow stream prior to mixing with the remainder of the screened waste flow.

The BOD and COD data also indicate increased removals when chemical flocculants were added. An exception to this observation is the COD removal data with and without chemicals for the 1969 overflows (Table 13). It is felt that the small decrease in COD removals was due to an increase in the dissolved organic fraction of the raw combined overflow. During late summer and fall leaves and other decaying organic material can cause a substantial amount of organic material to be converted to the dissolved state. This was very evident in run number 6919 as the appearance of raw waste after filtering was quite yellow. Run numbers 6928, 6929, and 6930 also had a very high dissolved fraction. This increase in dissolved organics resulted in the small decrease noted when chemical flocculants were added, since dissolved organic removals can be accomplished only by precipitation. Apparently, the dissolved organic material was not exerted as BOD (5) possibly due to unacclimated organisms and BOD removals were therefore not affected to the same degree as COD removals.

A summary of removal efficiencies for particulate organic material is presented in Table 14. It may be seen that during the 1969 runs, numbers 6923 through 6929, particulate COD removal ranged from a high of 96% to a low of 34%. It is obvious from the spread of the data that effective coagulation was not always being achieved. During the 1970 runs (700 Series) particulate COD removals were much more consistent reflecting the improved chemical treatment system, i.e., ferric chloride and polyelectrolyte addition. The average particulate COD removal efficiency for the 1970 data was 76±8% at the 95% confidence level. This value compared closely to the suspended solids removals for the 1970 data of 71±9% (Table 13) thus indicating that effective chemical treatment was generally obtained during the 1970 overflows. There are, however, some overflows (7012, 7014, 7015, 7023) where the particulate COD removals were below the 95% confidence range. This further illustrates the need for some type control system for metering the chemical addition.

Removal of dissolved organic compounds can only be obtained by precipitation of the material or adsorption on the flocculated particles obtained when polyelectrolyte or other flocculating chemicals were added to the raw waste stream. Removal of dissolved COD for those runs, where data is available, is presented in Table 14. As expected removal of dissolved COD was erratic and ranged from no removal to a high of 43% removal. Generally, the removals were in the range of 20 to 25%. Other chemical flocculants may give slightly improved removals, but 20-30% removal of dissolved organics is all that can be expected in a screening/flotation system.

It has been determined from laboratory studies that a flocculation period may further improve the removal efficiency of the flotation process. Research is now underway to substantiate these laboratory results at demonstration system scale.

TABLE 14

SUMMARY PARTICULATE AND DISSOLVED ORGANIC REMOVAL EFFICIENCIES

Run #(1)	Chemical Dosage C-31 mg/l	FeCl$_3$ mg/l	Clay mg/l	Particulate COD Removal %	Dissolved COD Removal %
6923	6.0	0	6	54	+2
6924	0	0	0	34	+14
6925	0	0	6	86	+25
6926	2.7	0	0	83	+22
6927	0	0	0	73	+23
6928	3.0	0	0	57	+23
6929	3.0	0	3	39	+23
704	6.0	0	0	63	+11
705	6.0	0	0	95	0
706	6.0	0	0	95	-2
707	6.0	0	0	74	+10
708	6.0	0	0	85	-3
709	4.2	30	0	88	+20
7011	5.3	17	0	87	+27
7012	3.9	16	0	53	-19
7013	4.6	16	0	80	+30
7014	3.5	21	0	42	+31
7015	4.5	21	0	50	+29
7016	4.5	21	0	93	+4
7018	4.5	21	0	87	+8
7019	4.5	21	0	77	+33
7020	3.8	18	0	81	+40
7021	3.8	18	0	91	+24
7022	0.5(2)	15	0	88	+43
7023	0.5(2)	15	0	48	+21
7024	0.5(2)	25	0	78	-3
7025	4.0	25	0	69	+22

1. Overflow Rate ∿2.5 gpm/sq ft
2. Herco Floc 810 used for these overflows

An important variable in the operation of a dissolved-air flotation system is the volume of pressurized flow required to supply sufficient air bubbles for proper process operation. For a given volume of liquid, the number of bubbles and their size will be a function of the operating pressure. Generally the greater the pressure, the larger the volumes of air dissolved and the smaller the size of the bubbles. There is of course a limit to the benefit to be gained by increasing operating pressure. Pressurized flow volumes generally utilized are in the range of 15-50% of the raw waste flow. Table 14 presents a comparison of pressurized flow ratios utilized. Rates from 13 to 22% of the raw flow were considered low values while rates from 23 to 44% were considered high values. The data is further segregated into those runs with and without chemical flocculant addition. A comparison of means test (68) was performed on the high and low pressurized flow values of Table 15. The procedure for performing this test may be seen in Appendix IV. The results of this comparison indicated there was no statistically significant difference between suspended solids removals for low or high pressurized flow whether or not chemical flocculants were utilized.

It appears considering all data, that a pressurized flow value of 20% of the raw flow will be sufficient for dissolved-air flotation operation on combined sewer overflow. A slight increase in pressurized flow may be required if the improved efficiencies predicted in the laboratory are obtained in the field, since a larger volume of solids will require floating. Air requirements based on a 20% pressurized flow system would be 1.4 SCFM per million gallons per day of raw flow capacity. An operating pressure of 50 psig provides sufficient air solution and a small enough bubble size to be effective without requiring excessive pressure drops at the pressure reduction value.

Another variable which can significantly affect the removal efficiencies of a dissolved air flotation system is the overflow rate, i.e., gpm/ sq ft of tank area. Removal data presented in Table 13 were all at the overflow rate of approximately 2.5 gpm/sq ft.

To allow for a study of higher overflow rates, the flotation tank of the demonstration system was partitioned as shown in Figure 17 for all 1970 runs. All process elements up to the point of entering the flotation zone were identical to the 1969 runs. A longitudinal baffle was placed the entire length of the tank. This had the effect of producing two separate flotation zones. Lateral baffles were then placed at one-half the length of the tank on side one and at two-thirds the length of the tank on side two. Effluent samples were taken just as the water flowed under these baffles to insure collection of a sample representative of the portion of the tank being utilized. The unshaded area of Figure 17 represents the unused portion of the tank. The raw flow as it enters the flotation zone is split into equal volumes. Since the effective length of each side of the tank is different, two overflow rates are thus obtained on the same storm. This completely eliminates the effect of any interacting variables like pressurized flow, chemical addition, and the

TABLE 15
COMPARISON OF PRESSURIZED FLOW RATES

Low Pressurized Flow

Run #	Pressurized Flow-%	Chemical Flocculents Utilized	Suspended Solids Removal-%
6911	17	No	30
6912	21	No	55
6914	13	No	80
6915	16	No	53
6917	16	No	64
6924	16	No	48
6927	18	No	61
7012	17	No	52
Average	16.8	--	55
6920	21	Yes	70
6921	17	Yes	57
6922	16	Yes	66
6923	20	Yes	52
6928	19	Yes	49
6930	19	Yes	40
707	22	Yes	68
708	22	Yes	76
7015	19	Yes	37
7016	20	Yes	85
7018	20	Yes	86
7019	20	Yes	71
7020	19	Yes	85
7022	19	Yes	85
Average	19.5	--	66

High Pressurized Flow

Run #	Pressurized Flow-%	Chemical Flocculents Utilized	Suspended Solids Removal-%
692	24	No	40
693	29	No	65
695	34	No	44
696	29	No	34
697	31	No	28
699	23	No	40
6910	23	No	32
6913	33	No	63
6917	23	No	64
6925	29	No	46
Average	27.8	--	46
6919	28	Yes	65
6926	31	Yes	75
6929	27	Yes	45
704	37	Yes	77
705	44	Yes	79
706	40	Yes	80
709	28	Yes	82
7011	24	Yes	69
7013	23	Yes	72
7014	28	Yes	68
7021	23	Yes	72
7023	25	Yes	49
7024	27	Yes	66
Average	29.6	--	69

NOTE: Overflow Rate @ ~2.5 gpm/sq ft

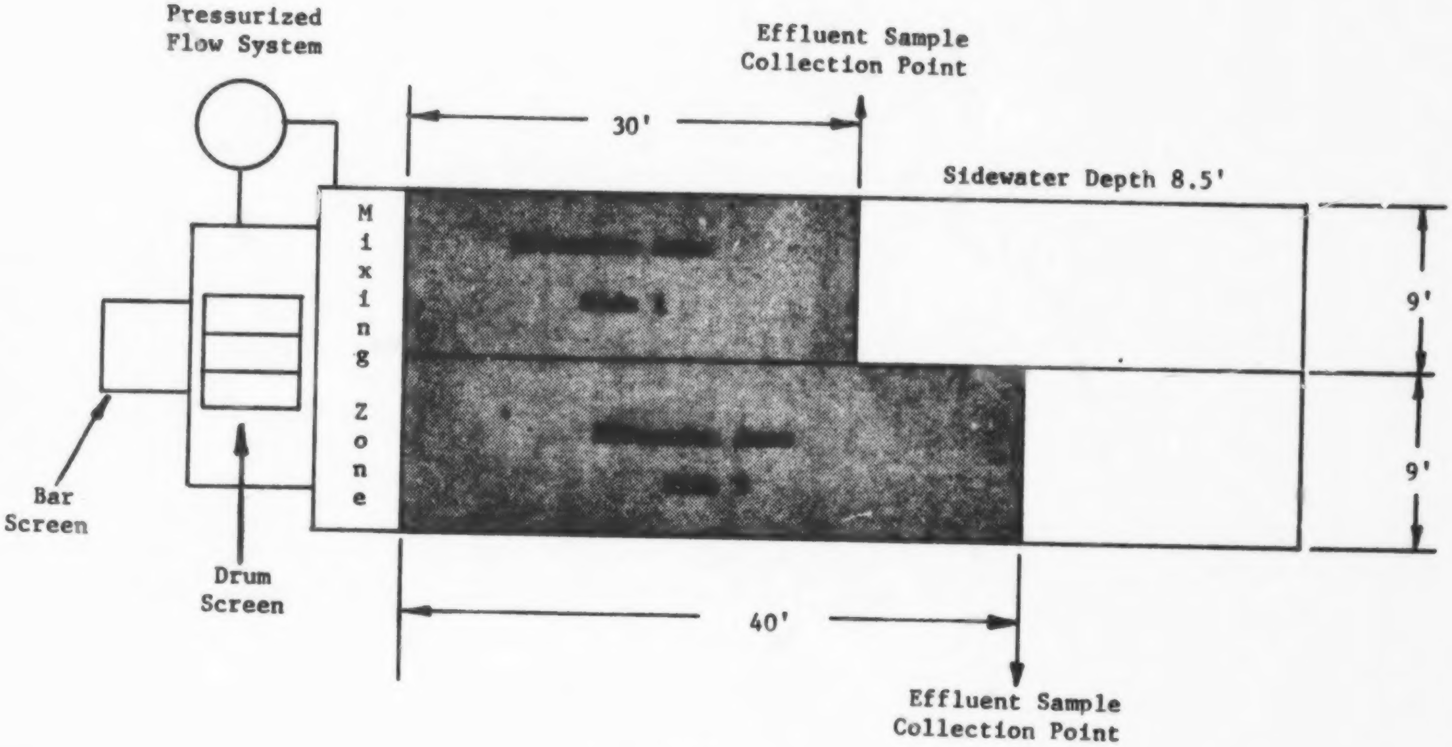

FIGURE 17

BAFFLE ARRANGEMENT FOR OVERFLOW RATE TESTS differences inherent in any two combined overflows, allowing direct determination of the effect of overflow rate.

Data taken on direct comparison of overflow rates are presented in Table 16. The data indicates removal efficiencies decrease as the overflow rate is increased. Data from each run is presented, as well as average values for low and high overflow rates along with the 95% confidence range. By comparing the mean values, removal efficiency decreased approximately 5-14% as the overflow rate increases from ~2.5 to 3.75 gpm/sq ft. A paired comparison test (Appendix IV) was also run on the data of Table 16. The results of this test indicated that there was a difference in the low and high overflow rate data. Confidence levels at which the difference was significant are as follows: COD - 90%, BOD - 95%, suspended solids - 99% and volatile suspended solids - 95%. It is anticipated that at higher overflow rates the decrease in removal efficiencies will be even greater. However, higher rates will be evaluated during this study. Based on the data of Table 16 an overflow rate of 3.3 gpm/sq ft is recommended as the basic design value.

Of considerable importance in the operation of a screening-flotation system is the volume of residual solids produced during operation. Volumes of floated scum generated during operation of the demonstration unit ranged from 0.75 to 1.41 percent of the raw flow at the 95% confidence level. Scum concentration was generally in the range of 1 to 2% solids on a dry weight basis (Appendix IV, Table IV-9). The floated scum and screen washwater from this project are disposed of via an interception sewer which directs them to the sewage treatment plant for ultimate disposal. Since gravity flow is utilized it is desirable to limit the sludge concentration to about 2% as this concentration easily flows by gravity. Higher concentrations can be obtained by skimming the floated scum blanket at less frequent intervals. Sludge concentrations as high as 4% solids have been obtained in this manner. Obviously the higher the concentrations, the smaller the volume of sludge produced for ultimate disposal. For this reason it is felt that in full scale operations sludge volumes will be somewhat less than those obtained during this study. Another factor, which will offset this reduction in scum volume, is the addition of more effective chemical flocculants. As the chemical flocculant addition is optimized, the volume of scum will increase due to the additional chemicals added and the higher efficiency of solids capture.

Ultimate solids disposal will dictate the desired solids concentration in the floated scum and, hence, the volume of scum which requires disposal. If scum is disposed of by way of an interceptor sewer a solids content in the range of 1-2% will be desirable. This should result in an average scum plus screen wash volume of 1.75 percent of the raw flow. If disposal is by tanker truck or solids dewatering at the site, a high solids concentration is desired (4%), and the volume should be less than 1 percent of the raw flow. Additional discussion on solids handling is presented in the full scale design consideration section of this report.

TABLE 16

COMPARISON OF THE EFFECT OF OVERFLOW RATE ON REMOVAL EFFICIENCIES [3]

Run	Low Overflow Rate ∿2.5[1]				High Overflow Rate ∿3.75[2]			
#	COD	BOD	SS	VSS	COD	BOD	SS	VSS
704	35	69	77	90	28	55	33	51
705	72	69	79	79	74	69	79	80
706	76	75	81	73	72	54	66	64
707	52	61	68	72	43	58	50	62
708	67	59	76	77	62	36	66	71
709	66	51	82	76	61	51	76	73
7011	63	60	70	76	60	64	74	79
7012	41	36	52	58	36	31	36	56
7013	56	55	72	80	48	46	66	69
7014	36	--	68	68	29	--	66	68
7015	43	37	52	47	39	40	48	27
7016	74	--	85	84	71	--	83	82
7020	74	77	85	86	68	45	63	69
7021	58	76	72	70	67	80	67	68
7023	37	51	49	49	46	46	35	36
7024	61	52	66	61	51	46	56	70
7025	50	62	59	61	60	64	68	70
Average	56.5	59.3	70.2	70.7	53.8	52.3	60.7	64.4
95% Confidence Limits	49-64	52-66	64-76	64-77	46-62	45-60	53-69	57-72

1. Actual Range 2.43 - 3.1 gpm/sq ft
2. Actual Range 3.4 - 4.5
3. Removal in %

NOTE: chemical flocculants added for all runs

During the operation of the demonstration system some settled sludge was noticed. The amounts of settled sludge were, however, extremely small, with one exception. Runs number 691 and 692 were made without using the drum screen. Upon draining the tank after run number 692, about 4-6" of settled material was discovered. This consisted of grit, twigs and other material which could not be floated. Later in the year the tank was again drained after 13 additional runs which were all made with the screen in service. Settled sludge volume amounted to only 2-3 inches at this draining. This clearly demonstrates the value of the screen in removing heavy particulate matter which cannot be floated in the flotation tank. The tank was again drained at the end of the 1969 season and sludge volumes were again only 2-3 inches. It may be concluded that bottom skimming systems are not required when screening procedes flotation. If, however, applications arise where flotation is used alone, it is imperative that some type of bottom skimming be provided or large sludge blankets will form which will eventually scour into the effluent.

Material balances for suspended solids were made for almost all runs. The tabulated data may be seen in Table IV-14, Appendix IV. The balance of solids in and solids out of the system generally totaled within about 25%. In some cases, however, the error was higher. The probable reasons for the material balance errors are believed to result from scum sampling and collection procedures. At the end of a run, the floated scum blanket which remained on the tank was lost with respect to material balances. Generally this scum was skimmed off after the run when the scum meter was not totalizing. It was also difficult to obtain a representative sample of scum, since scum was scraped intermittently and the solids concentration could vary widely from the beginning to the end of the skimming cycle. Scum samples were taken manually and if other operating problems arose, a scum sample from every skimming cycle may not have been obtained. Because of the sampling procedures it is felt that scum suspended solids concentrations were not representative. Since suspended solids in the scum could account for a significant portion of the solids balances, it is felt that these errors contributed to a large extent in the material balance discrepancies.

Difficulties were also experienced with the raw flow meter totalizer. The units on the totalizer were 30,000 gallons per count and thus could not be read accurately. The totalizer has subsequently been changed and thus this problem eliminated. Another possible source of material balance error is the fluctuating concentration of the raw overflow. Limited data has been collected to indicate that the pollutant concentration can vary significantly during an overflow. This variation could introduce errors in the material balance.

The maintenance of the demonstration system has been limited to lubrication of moving components. The following discussion will be limited to those problems which were encountered and the method of solution. Some difficulties were encountered with the float switches utilized in the system. These switches are the reed type which are activated by a sliding float. They required frequent cleaning, once per week, as sand would work its way between the float and the shaft and caused the float to become inoperable. Replacement of this type switch with a conductivity type should solve this problem.

Difficulties were also encountered with the back pressure control valve on the pressurized flow system. This problem resulted from excess moisture in the compressed air. The valve positioner apparently could not tolerate the moisture level encountered. All problems were in the positioner for the valve which accepts a 0-20 psi control pressure and positions the valve accordingly. The positioner was utilized only because of the design flexibility required in a demonstration system. In the full scale applications when the pressurized flow rates are known, valve positioners will not be required and the problems incurred will be eliminated.

A minimum amount of difficulty was experienced with the chemical metering pump, but this was a result of improper materials selection and the difficulties were easily eliminated. As may be concluded from the above discussion, the operation problems encountered were minimal and this attests to the soundness of the system design.

Disinfection of Combined Overflows

Another important aspect in the treatment of combined sewer overflows is adequate disinfection. The screening/flotation system provides sufficient detention time (12-20 minutes) to obtain the necessary chlorine contact time. The system also is flexibie in that various points of chlorine addition are available depending upon process needs. A summary of the disinfection data taken during the project is presented in Table 17. Hypochlorite salts were utilized as a source of chlorine. Dosage was held essentially constant at 10 mg/l. It was noticed, however, that the strength of the chlorine stock solution could decrease rapidly depending upon environmental conditions. Therefore, some of the storms could have had dosages less than 10 mg/l. In general, relatively good disinfection was obtained. A trend seems to exist in Table 17 which indicates that as the coliform density increases the absolute coliform density in the effluent also increases. Nevertheless good disinfection can be obtained in conjunction with the operation of a screening/flotation system.

Conceptual Design

This section of the report deals with the design concepts necessary to utilize a screening/flotation system on a full scale basis for treating combined sewer overflow. It is not the intention of this discussion to provide specific answere to all design details. Sufficient information will be presented to form a basis for process design. Various engineering considerations and/or judgements must be provided to produce the final overall system design. It should also be recognized that this project is still underway and that later data may indicate changes in design philosophy.

TABLE 17

SUMMARY OF DISINFECTION DATA

Run No.	Chlorine Dosage mg/l	Point of Addition	Detention Time min	Influent Coliform per 100 ml x 10^5	Effluent Coliform per 100 ml
695	10	PF	20	36	<100
696	10	PF	20	5.7	<4
697	10	PF	20	1.3	<4
698	8	PF	20	7.8	<4
699	10	PF	20	6.2	2
6910	10	PF	20	19	<2
6911	10	PF	20	20	10
6912	10	PF	20	65	<5000
6913	10	PF	20	38	<5000
6919	10	EFF	10	310	60,000
6920	10	EFF	10	160	40,000
6921	10	EFF	10	55	<7
6922	10	EFF	10	82	15000
703	10	INF	21	270	<1000
704	10	INF	21	12.2	<100
706	10	INF	21	0.7	200
707	10	INF	21	340	3800
708	10	INF	21	110	18000

PF = Chlorine added in pressurized flow line
EFF = Chlorine added to effluent from flotation basin
INF = Chlorine added to raw waste prior to bar screen

Note: Samples were dechlorinated when necessary using Na_2SO_3.

The overall system can be divided into various subsystems and these subsystems are listed and discussed below:

1. Pumping system
    a. Self cleaning bar screen
    b. Pump pit
    c. Variable rate pumping system
2. Screening/flotation system
    a. Drum screens
    b. Pressurized flow system
    c. Sludge collection
    d. System tankage
    e. Flocculators
3. Solids slurry storage
    a. Mixers
    b. Transfer system
    c. Dewatering (if required)
4. Chemical Addition
    a. Chemical storage
    b. Mixing system
    c. Metering pumps
    d. Disinfection
5. Control Systems

Pumping System

The pumping system is obviously a vital subsystem in the overall treatment concept. It is envisioned that a sump will be utilized to provide the necessary pump suction conditions. A self cleaning bar screen will be utilized just up stream of the sump to remove large objects which will not pass the pumps and/or could cause problems down stream in the treatment system in such areas as the drum screens or sludge removal systems. Bar spacing is recommended at 1/2" as this value proved adequate in the demonstration system. A medium duty bar screen will have sufficient strength to handle the imposed solids loadings.

The pumping system must be able to handle the variable rates which will be encountered during operation. The pumps therefore must be controlled automatically to vary the pumpage as the raw flow varies. There are numerous pumping system configurations of this type which have been employed in the past for storm water pumping, municipal waste treatment and industrial processing. Many pump manufacturers offer package pumping systems (69) which would meet the required design consideratins. Therefore no detailed information on pump station design will be presented herein.

It should be noted however that open type screw pumps seem to be ideally suited to combined sewer overflow treatment systems, since they can provide variable flow rate from a constant speed motor. Since any system must be designed for a specific hydraulic flow rate, an overflow structure should be provided for the pumping system. Considerations involved in overflow design should include utilization of the sewer system for maximum storage of raw overflow. This will provide a damping effect on the raw flow rate, and minimize control problems in the treatment system. Proper design of the overflow will provide satisfactory treatment system operation with a minimum of excess overflows. Provisions should also be made for sump cleaning, since large volumes of gravel and grit are anticipated. Overall pump system design should provide for ease of cleaning and maintaining the pumps. No problems were encountered during operation of the demonstration syste, with regard to feed pumps. The demonstration system, however, had only a single feed pump and the flow rate did not vary during operation. Since the screening flotation system can function at overflow rates higher than the design value, the pumping system should be designed with this fact in mind and sufficient capacity should be provided. A pumping capacity of one and one-half the flotation design capacity should be provided, since the flotation unit can handle this additional volume for short periods with only moderate decrease in removal efficiencies.

It may be possible in some applications to utilize gravity flow. In this case only sluice gates will be required to direct the raw flow to the various screening/flotation modules. The gates should be automatically positioned to provide proper control. Consideration of head loss across the screen becomes more critical when contemplating a gravity feed design. It may also be possible in some applications to utilize gravity feed and effluent pumping. This approach should provide the most maintenance free pumping system since the effluent water will be low in suspended solids. When gravity feed systems are utilized, a sump or stilling basin will still be required to provide proper hydraulic control, and the bar screen is also required for system protection. Grit accumulations are expected to be greater in the gravity flow systems, since turbulance from the pump suction will not be present.

Screening System

The screening system consists of basically two sections, the drum screen and the backwashing system. The critical design parameters associated with the drum screen include hydraulic loading, solids loading, and head loss capability. The hydraulic loading is a function of the wetted screen area. This area varies only slightly as the head loss across the screen increases during operation. The hydraulic loading is not affected by rotation speed. Generally a drum screen can operate with up to 70% of the screen surface submerged. Higher submergence is not recommended due to possible flooding of the backwash water removal hopper. Hydraulic loading in the range of 25-45 gpm/sq ft of screen area was utilized in the demonstration system for a 50 mesh, 297μ opening screen media.

A hydraulic loading of 40 gpm/ft^2 and a screen size of 50 mesh are recommended at this time. Additional data on screen mesh size and hydraulic loading are being obtained in Phase V of this project. Upon completion of this phase, extensive information on screen design will be available.

Drum rotation speed controls the solids loading on the screen. It was found during this study that a solids loading of 1.2 pounds of dry solids removed per 100 sq ft of screen media produced a head loss of about 13 inches of water. Drum rotation speeds were in the range of 1 to 5 rpm. Solids loading may be calculated with the following equation:

$$L_s = F_s R / r A_D$$

where:

L_s = Solids loading per 100 sq ft

R = Screen removal efficiency (%)

F_s = Feed solids into screen (lbs per min)

r = Drum rotation speed rpm

A_D = Total surface area of screen

As may be seen from the equation solids loading is directly proportional to solids removal efficiency and inversely proportional to drum rotation speed. A removal efficiency (R') of 35% is recommended based on the results obtained in the demonstration unit. Drum rotation speeds of 2 to 12 rpm are also recommended.

It is apparent that selection of drum area is controlled by two criteria; hydraulic loading and solids loading. Either of these two variables may control the screen area requirements. If a solids loading of 1.2 lbs per 100 sq ft is utilized resulting head losses will be in the range of 12 to 14 inches of water. Higher head losses and subsequently higher solids loadings may be possible without a significant effect on process efficiency. Data to support this statement are now being obtained. The inside of the drum should be fitted with an angle iron system (similar to a roto-dip feeder), which will pick up any solids which will not adhere to the screen media and would not be carried up to the spray washing system.

The screen cleaning system consists of a pump, a header system and spray nozzles. A hopper inside the drum collects the material as it is flushed from the screen. Required screen wash water rates amount to 15 gpm per foot of drum length. This volume is sufficient to clean a completely blinded screen. The spray nozzles utilized should provide a low mechanical pressure on the screen media, but still provide good washing capability. The nozzles should have as large an opening as possible consistant with a good spray pattern. The nozzles utilized on the demonstration unit had egg shaped openings with the smallest dimensions being 1/4 of an inch. They were positioned about 12" from the drum surface and provided excellent cleaning capability. They exerted an average pressure of less than 1 psi on the media.

Some problems were encountered with nozzle plugging due to an inefficient seal between the raw and screened water. Plugging occurred mostly from gravel or grit with some plugging due to string and twigs. The drum seal has subsequently been improved, but to eliminate a possible plugging problem, a small wet cyclone (six inch diameter) should be utilized to trap any material which could cause plugging. The dirty water discharge from the cyclone should be routed back through the screen.

It is recommended that screen rotation be continuous. This eliminates rapid change in head loss when the clean screen enters the water, which could result in an incomplete washing cycle. If the drum were stationary, the portion of media not in contact with the flow would initially be clean. As the head loss reached the limit of the backwash actuating system and the drum was accivated, it would rotate only until sufficient clean screen area was wetted to reduce the head loss to normal values and the washing system would then be deactivated. This could leave portions of the screen unwashed and cause rapid head loss flocutations. When the rotation is continuous, a solids layer gradually builds up over the entire drum surface. The back wash system is then activated and cleans the entire screen before deactivating. It is recommended that the spray water cleaning system be activated by monitoring differential pressure, i.e. head loss across the screen. Conductivity probes can be utilized, and there are a number of commercially available, inexpensive units on the market. The system consists of two probes and an electrical circuit board with relays. The probes are mounted in the tank at the desired differential elevation. When the water contacts both probes, a relay is tripped which can be used to activate the backwash pump. The relay remains closed until the water level is below the lower probe. The differential chosen should be about 50-60% of the expected head loss across the screen, i.e. for a 2' head loss the spray system should activate at 1 to 1.2 foot of head loss. This prevents the screen from becoming completely blinded prior to activation of the back wash system. The lower probe should be set two to three inches higher than the head loss level when the screen is completely clean. This will prevent unnecessarily long back wash cycles.

The media utilized in the demonstration system was type 304 stainless steel. This media proved adequate. Because of the near neutral pH encountered in the demonstration runs (6.8 - 7.5), brass screen which is considerably less expensive could be utilized. During the two years of operation reported herein some bacterial growth on the screen has been experienced. It was found that a rinsing with chlorine solution would remove these growths.

The control of the feed water flow into the drum screen is extremely important. Since the possibility exists, with this highly variable raw waste, that the head loss capacity of the screen may be exceeded during the heavy solids loading peaks. The system should therefore have an automatic by-pass feature, which will allow relief of the screen if the head loss capacity is exceeded. The water which is bypassed during these times can be routed to the flotation zone. It is also recommended that a head loss capacity of 24 inches of water be provided.

Figure 18 presents a sketch of the recommended screen configuration. This sketch illustrates the important features of the screen installation which have been discussed above.

Flotation System

The design of the flotation system requires the consideration of the following components: pressurized flow system, flotation basin and scum removal system. Details on design recommendations will be discussed in this section along with a proposed system arrangement.

The pressurized flow system is the heart of the dissolved air flotation process. It includes a pump, air solution tank, pressure reduction valve, source of compressed air and suitable control systems.

The pressurized flow system should be designed to provide a volume equal to 30% of the raw flow rate into the system. This should provide an adequate margin of safety since testing at the demonstration site indicated 20% was sufficient. The recommended design operating pressure is 50 psig. The air solution tank should provide maximum air water interface to obtain high air solution. Some air solution tanks are packed with a tower packing to provide this interface. For treatment of combined overflows, however, a packless tank is recommended. A packed tank would be very susceptible to plugging due to the solids present in the waste stream. Packless tanks are generally fitted with an internal baffle to promote air water interface. Nominal detention time in the tank is generally in the range of one minute. The tank should also have some method of controlling the water level, since only about 20% of the tank volume is filled during operation.

The pressure in the system is controlled via an adjustable valve. Weir type valves have been utilized successfully. The valve is positioned by way of a pneumatic controller which allows automatic control of the system pressure. The valve besides providing pressure control, provides the necessary shear forces to promote proper bubble formation.

The sizing of the flotation basin is based mainly on the American Petroleum Institute (API) standards (64). Based on operation of the demonstration system, some modifications have been made to this procedure. The API design criteria are listed below as follows:

$$L = 1.2\ FV_hd/V_t$$

where: L = effective tank length (ft)

V_t = rate of rise of particles (fpm)

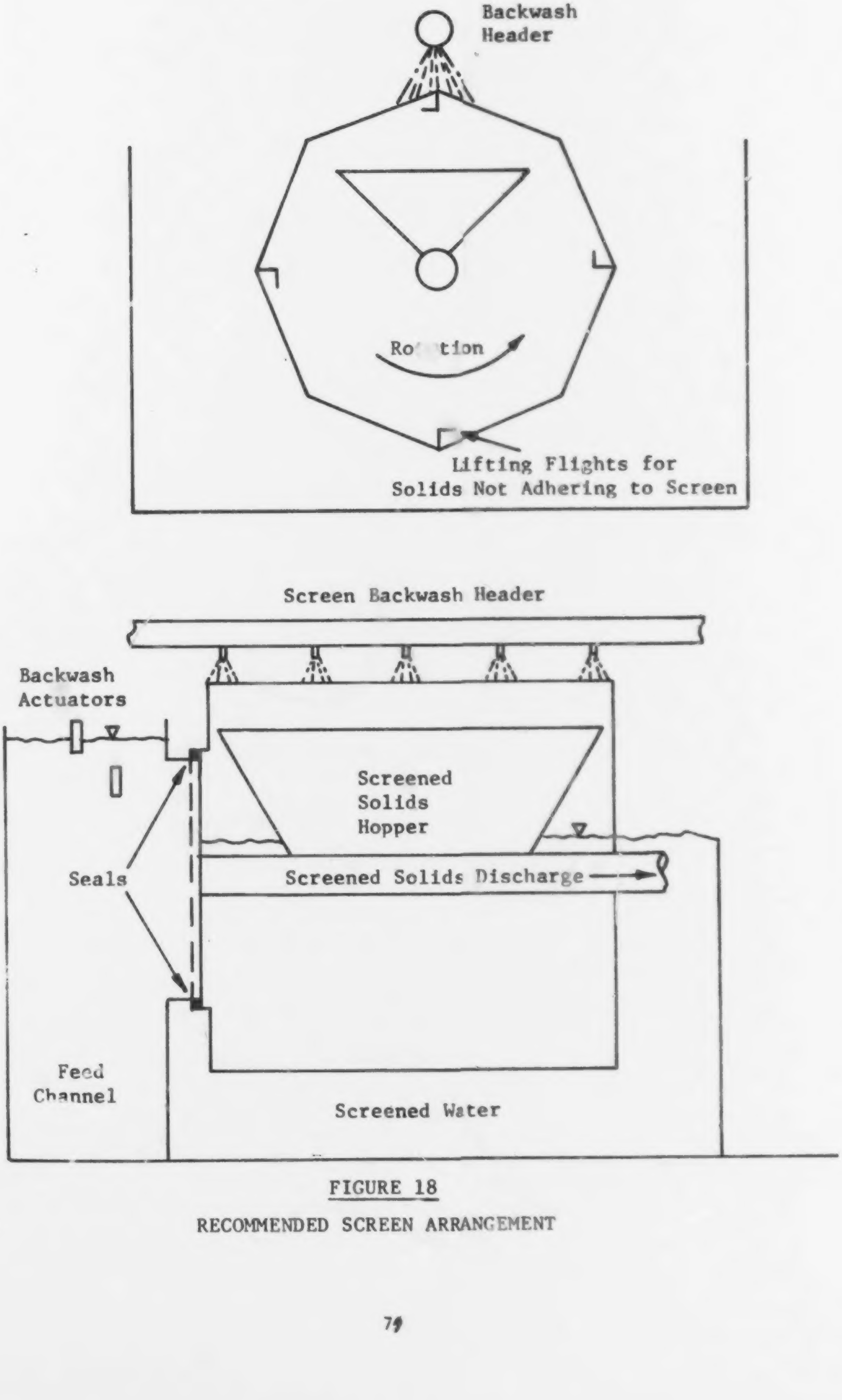

FIGURE 18

RECOMMENDED SCREEN ARRANGEMENT

V_h = horizontal velocity in tank (fpm)

d = effective tank depth (ft)

F = $(0.026\ V_h/V_t) + 0.995$

V_h maximum = 15 V_t or 3 fpm

t_D (tank detention) = 10 minutes minimum

d (depth of tank) = 0.3 to 0.5 of tank width

minimum depth = 3'

The particle rise rate can also be expressed as a surface loading rate, i.e.,

$$S_L = (V_t)7.48$$

where S_L = surface loading gpm/sq ft and V_t is in fpm

The demonstration system design was based on the API procedures listed above. A particle rise rate of 0.4 fpm (S_L = 3 gpm/sq ft) was utilized based on laboratory tests. The tank was also designed to allow changing the overflow rate. It is obvious from the above discussion that the particle rate of rise is critical in the flotation basin sizing. It may also be seen that the horizontal velocity will be at the maximum value (3 fpm) for all rise rates in excess of 0.2 fpm. The particle rise rate is dependent upon particle diameter to the second power and the apparent difference in density of the particle bubble combination and suspending fluid to the first power. Little can be done to control the latter, but the particle size can be significantly affected by proper chemical addition and flocculating procedures. Rise rates as great as 5 fpm have been obtained in the laboratory, indicating large floc particles can be developed.

It is recommended that the above discussed procedures be utilized to size the flotation basin. A particle rise rate of 0.45 (S_L = 3.3 gpm/sq ft) should be utilized. This recommendation may be modified, depending upon results obtained during the remainder of the project.

Overhead skimmers are provided to remove the floated scum. Bottom skimmers are sometimes utilized on flotation systems to remove any sludge which may possibly settle. If screening (50 mesh or finer) is utilized in the system, bottom skimming is not recommended since the small amount of settled sludge expected can be removed while draining the tank between storms. If, however, flotation is utilized without screening, bottom skimming will be required. Removal of scum should be controlled by a timed cycle or the sensing of a sludge blanket. This will allow sludge to be removed only when required and hold to a minimum the volume of floated scum which will require ultimated disposal.

The recommended general arrangement for the screening/flotation system is presented in Figure 19. It is felt that this configuration will meet all process requirements and be the most economical system.

The additional subsystems required to complete the treatment system are sludge storage and the chemical storage and feeder units. A possible overall site arrangement including these subsystems is shown in Figure 20. Sludge storage should be sufficient to handle 2% of the raw flow for the design storm period. This should provide sufficient storage volume, however, the ultimate sludge disposal method may dictate the desired storage volume. Sludge disposal alternatives available include tanker trucking, providing a truck mounted vacuum filter to reduce the slurry to a cake for hauling, or disposal by pumping to an interceptor sewer for transporting to the sewage treatment plant. If the latter method is used, the interceptor obviously must not have possible overflow points in its course to the sewage treatment plant. If disposal via interceptor is utilized a sludge pump will be required and essentially no solids storage pit will be necessary. On the other hand, if it is not feasible to pump solids into the interceptor during a storm, they may be stored at the site and pumped to the interceptor after the storm has passed. Regardless of which method is selected, proper attention should be given to the problems associated with handling sewage solids slurries.

The chemical treatment module (Figure 20) should provide for storage of flocculating chemicals (ferric chloride and a polyelectrolyte) as well as chlorine. A sufficient supply of chemicals consistent with the requirements of the system should be provided. Estimated chemical dosages are: ferric chloride - 20-25 mg/l, polyelectrolyte - 5 mg/l, and chlorine - 10-15 mg/l. The chemical treatment module also houses the proper feeder systems to accurately dispense the chemicals into the wastewater flow.

The entire system detailed in Figure 20 should be automated 100%. This will allow remote monitoring and control of the treatment facility at a central location. However, the maintenance involved with automatic systems which are used intermittently will have to be tolerated. It is felt that this maintenance will not be of such a magnitude to negate the advantages of automation.

Economic Considerations

There are many factors which must be considered when estimating the capital costs of a combined sewer treatment facility. The basic areas of consideration for a screening/flotation system are listed below:

1. Screening/flotation system including
    - self cleaning bar screen
    - variable rate pumping system

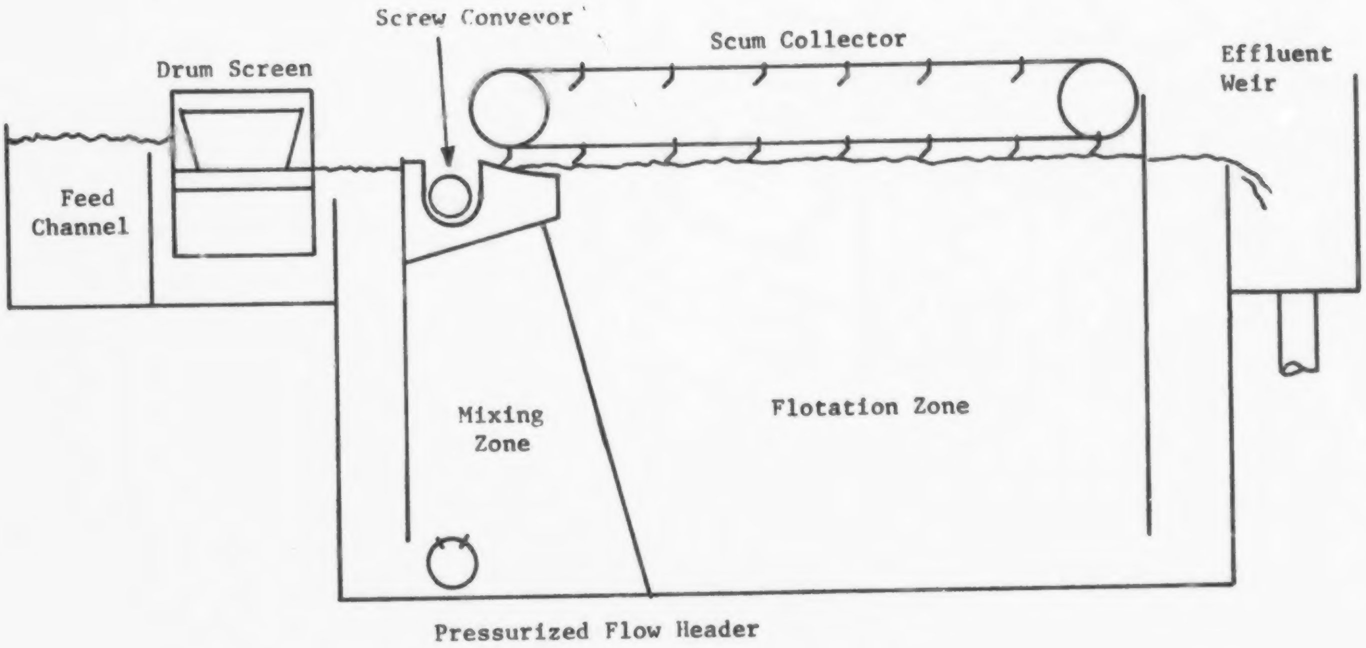

FIGURE 19

RECOMMENDED SCREENING/FLOTATION ARRANGEMENT

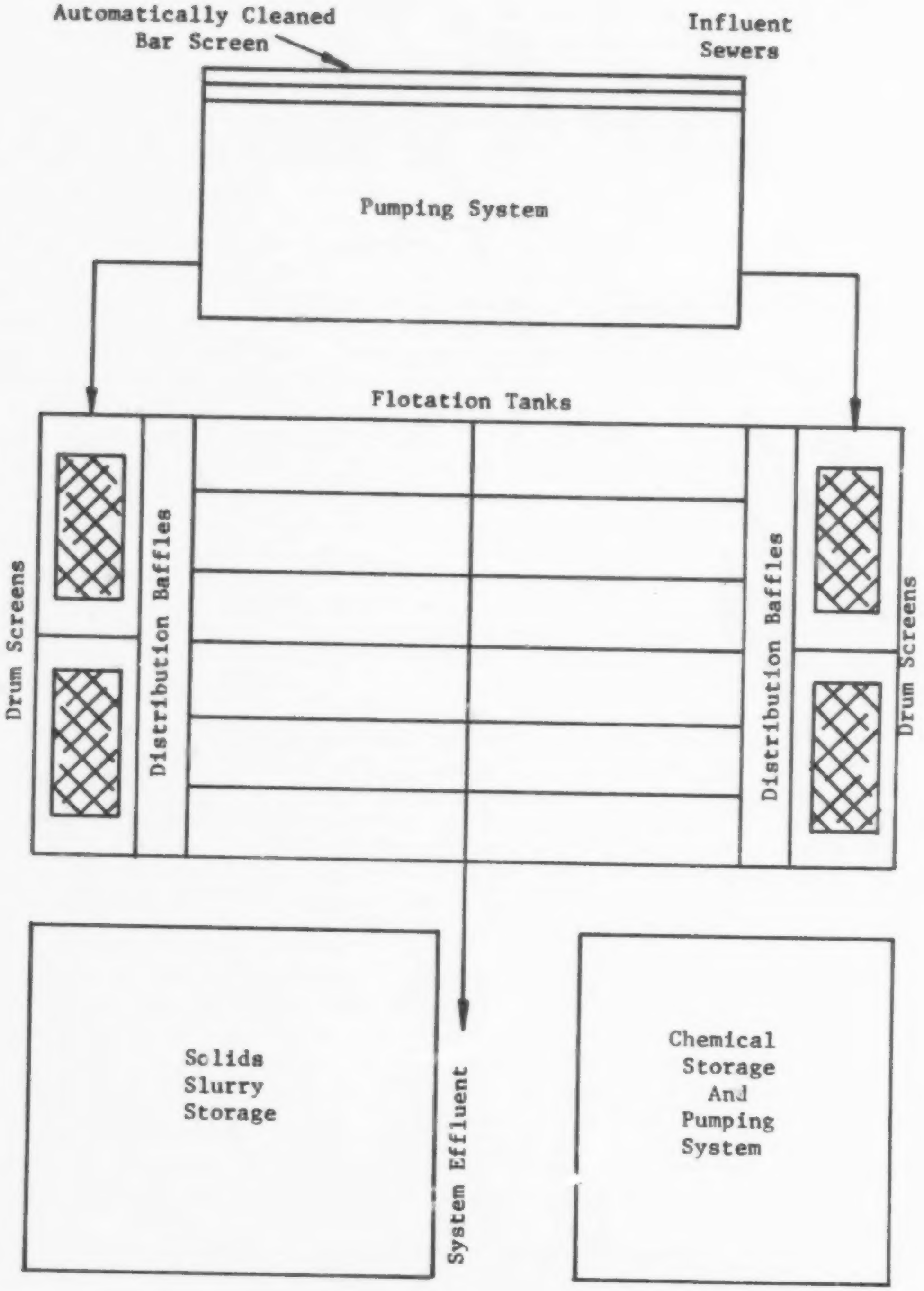

FIGURE 20
OVERALL SYSTEM CONFIGURATION

solids storage and disposal
instrumentation
control building
erection costs

2. Sewer interconnection (i.e. combining a number of overflow points to reduce the number of treatment sites) and outfall facilities
3. Land costs
4. Special design problems
   foundations (soil problems)
   ground water
   special construction techniques
   freezing problems
5. Engineering costs and fees

Generalized cost estimates for Item 1 can be made since these costs are now known. Cost estimates for Items 2, 3, 4, and 5 are highly specific for the selected treatment sites, and cannot be generalized. Total installed costs for the screening/flotation system is estimated at $19,000 per MGD capacity for 10 MGD plants and $15,000 per MGD for 45 MGD and larger plants. This price includes system tankage in concrete, all necessary hardware, pressurized flow system, variable rate pumping system and scum collection and storage. The foundation is assumed to be a simple slab, and solids disposal by gravity drainage into an interceptor sewer.

The total installed cost for the demonstration facility was approximately $90,000 or $18,000 per MGD capacity for the 5 MGD plant. It should be noted that the demonstration system did not have a self cleaning bar screen, a variable rate pumping system, or solids storage facilities which would increase the cost per MGD capacity. On the other hand the demonstration system was of all steel construction and only 5 MGD capacity, larger capacity and concrete construction are expected to decrease the cost per MGD capacity.

Based on the above generalized information, the cost of providing a screening/flotation system to treat the entire Hawley Road drainage area (∿500 acres) is presented below. It was assumed that the design storm was the once in 5 year storm. From Milwaukee rainfall records this intensity is approximately 1.0 inch per hour for the estimated time of concentration of the drainage area of 100 to 110 minutes. This amounts to an overflow rate of approximately 120 MGD. Allowing a 33% surge factor for this peak storm results in a design capacity of 90 MGD. The following costs are estimated for this system capacity.

1. Screening/flotation system	$1,350,000
2. Sewer interconnection and outfall facilities	$ 65,000
3. Land costs (1.5 acres @ 100,000/acre)	$ 150,000
4. Special design problems	$ 90,000

5. Engineering costs and fees	$ 240,000
Total	$1,895,000
Cost/MGD	$ 21,056
Cost/acre	$ 3,828

A detailed design and cost estimate for a midwest city indicated a system cost of $22,000 per MGD capacity which compares favorably with the above estimate for the Hawley Road drainage basin.

The operating costs for a screening flotation system include power, chemicals, and maintenance. Table 18 presents a summary of the estimate operating costs. The total operating cost is 3.09¢/1000 gallons of treated overflow. The majority of this cost is for chemical addition, *i.e.* 2.51¢/1000 gallons. These costs are based on operation of the demonstration system at Hawley Road and are not expected to vary significantly with plant size.

TABLE 18

OPERATING COST ESTIMATES

Item	Quantity	Unit Cost	Cost ¢/1000 gallons
Power	15 KW/MGD	1.5¢/KWH	0.54
$FeCl_{3(1)}$	20 mg/1	4.5¢/1b	0.75
Polyelectrolyte(1)	4 mg/1	35¢/1b	1.17
Chlorine(1)	15 mg/1	4.7¢/1b	0.59
Labor(2)	30 hr/mo	$10/hr	0.02
Parts(2)	$300/mo	---	0.02

TOTAL COST = 3.09¢/1000 gal

(1) Based on carload prices

(2) Based on 90 MGD system 40 hours per month operation

SECTION VIII

ACKNOWLEDGEMENTS

The cooperation of the City of Milwaukee on this project is deeply appreciated. Messrs. Herbert A. McCullough, City Engineer, and Gilbert Vosswinkel's assistance in site selection and review of construction plans allowed timely completion of the construction of the demonstration facility.

Many people at the Rex Ecology Division contributed to the success of this project. Design of the demonstration system was made by J. E. Milanowski. Operation of the unit at any time day or night was performed by M. K. Gupta, F. Toman, and D. G. Mason. Excellent cooperation was provided by the process equipment laboratory, headed by R. W. Wullschleger, with regard to timely completion of the large numbers of analyses which were required.

The principal author of the report is Donald G. Mason. Valuable assistance in preparation of the literature search and the chemical oxidation studies sections of the report was provided by M. K. Gupta.

Encouragement and assistance from EPA personnel, i.e. Messrs. William A. Rosenkranz, Frank Condon, Ralph Christensen and Clifford Risley is also deeply appreciated.

SECTION IX

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SECTION X

PUBLICATIONS

The data reported in this report have been utilized as a basis for three technical papers:

1. Mason, D.G., "The Use of Screening/Dissolved-Air Flotation for Treating Combined Sewer Overflows," Presented at the Seminar on Storm and Combined Sewer Pollution Problems, Edison, New Jersey, November 4-5, 1969.
2. Mason, D.G., "Screening/Dissolved-Air Flotation for Treating Combined Overflows," Presented at the Seminar on Combined Sewer Overflow Abatement Technology, Chicago, Illinois, June, 1970.
3. Mason, D.G., "Treatment of Combined Sewer Overflows Utilizing Screening/Dissolved Air Flotation", Presented at the WPCF Conference October 3-8, 1971, San Francisco, California

APPENDIX I

CHEMICAL OXIDATION STUDY

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LITERATURE SEARCH

Removal of organic contaminants from combined sewer overflows cannot be effectively and economically accomplished by conventional biological oxidation methods (1)(2) for the following reasons:

A. Large and highly variable flows
B. Intermittent operation would be required because of the nature of the flows, and this has a very detrimental effect on biological oxidation systems (3)(4)(5).
C. Removal of the solids produced would require a large number of sedimentation tanks (because of high flows) which would be used only periodically.

Use of chemical oxidation on combined overflows has the potential advantage of effective destruction of biological organisms and the removal of organic contaminants without the production of residual waste concentrates. Some of the technical difficulties connected with the successful treatment of wastes by chemical oxidation include the relatively dilute concentrations of organic materials present, the unknown composition of a wide range of possible organic compounds present, and the continually changing concentrations and compositions present in a combined wastewater flow such as the one with this project is concerned.

These difficulties necessitate the use of gross parameters in evaluating the effectiveness of a given chemical oxidant. Such parameters would include knowing the chemical oxygen demand of the waste (COD) and the concentration of the oxidizing properties in the chemical being utilized. Having this information, the oxidation efficiency (6) can be calculated as shown below:

$$\text{Oxidation Efficiency} = \frac{\Delta \text{COD}}{\text{Available Oxidation Equivalents} \times 100 \text{ (6)}}$$

Where: ΔCOD = change in COD brought about by the oxidation process (in milligrams of O_2 per liter)

And: Available Oxidation Equivalents = amount of chemical oxidation equivalents available from the oxidant used (in milligrams of O_2 per liter)

Taking into account the variation in organic concentration and volume of the waste flow to be treated under this contract, the "ideal" oxidant should have the following characteristics:

A. It must be nonspecific in its attack on organic materials.
B. Its required contact and retention time should not be excessive (30 minutes or less).
C. It should be easy to handle and dispense.
D. It should effectively oxidize the organic materials at the prevailing pH of the waste
E. It should not produce any secondary pollutants which are difficult to remove from the waste flow.

From the literature (6) it was found that six types of oxidation systems offer various degrees of potential for the treatment of organic impurities in wastewater. These systems are:

- A. Oxidation by oxidants containing active oxygen
- B. Accelerated molecular oxygen oxidation
- C. Catalytic oxidation of adsorbed organics
- D. Oxidation by chlorine and its derivatives
- E. Oxidation by oxy-acids and their salts
- F. Electrochemical oxidation

Of the six oxidation systems mentioned, only systems A, D, E, and F appear to have potential for treating combined sewer overflows. These systems will be discussed in detail below.

Hydrogen Peroxide

Considerable work has been done regarding the application of hydrogen peroxide (H_2O_2) as an oxidizing agent in the treatment of wastewater containing organic impurities (6)(7)(8)(9)(10)(11)(12).

In aqueous media, hydrogen peroxide decomposes to form the ·OH radical. This radical is one of the strongest oxidants known in aqueous systems. However, hydrogen peroxide does not, when used alone, oxidize organic materials within a practical reaction time. The system requires a multivalent iron salt as a catalyst.

The principle reactions involved in a hydrogen peroxide iron salt system are as follows (11)(12):

1. $Fe^{+++} + H_2O_2 \rightarrow Fe^{+++} + HO_2 + H^+$

2. $Fe^{++} + H_2O_2 \rightarrow Fe^{+++} + OH^- + {}^{\cdot}OH$

Considering reactions 1 and 2, above, the iron salt acts truly as a catalyst i.e., is not utilized during the reactions. Reaction 1 is the rate limiting reaction. Once the ferrous ion is formed, reaction 2 is extremely rapid (11)(12). The production of ·OH radicals via reactions 1 and 2 using ferric salts requires two to three hours at 65°C (6). By using ferrous salts, the production of ·OH radicals is essentially immediate, but the iron salt no longer performs as a true catalyst, since it is not reproduced at the same rate at which it is being utilized. There are other reactions associated with the H_2O_2--iron salt system which compete with the organic material being oxidized for the ·OH radical. The following reactions illustrate the competition for ·OH radicals (11)(12).

3. $Fe^{++} + {}^{\cdot}OH \rightarrow Fe^{+++} + OH^-$

4. $HO_2 + \rightleftarrows O_2^- + H^+$

5. $Fe^{+++} + O_2^- \rightarrow Fe^{++} + O_2$

6. $H_2O_2 + {}^{\cdot}OH \rightarrow HO_2 + H_2O$

Hence, only an unknown portion of the $^{\cdot}OH$ radicals produced are available to oxidize the organic substrate.

The efficient production of $^{\cdot}OH$ radicals requires a specific pH range of 3 to 5 (6)(7)(10)(11). Above or below this specific pH range, production of $^{\cdot}OH$ radicals is greatly reduced. During the reactions, an excess of H^+ ions are formed (9) so the pH will normally stabilize in the optimum range. Eisenhauer (7) found that the initial pH of an ABS solution could be as high as 11, before it would significantly affect pH stabilization in the 3 to 4 pH range. The pH of the system can also be lowered to the specific range by use of acid (11).

The reactions between $^{\cdot}OH$ radicals and specific organic substrates were summarized by Busch (12) after review of a number of papers on the subject. Since the waste to be oxidized under this contract contains many different types of organic substrates in unknown proportions, these specific equations were not directly applicable and hence were not reproduced herein.

Use of the hydrogen peroxide method of oxidizing organic wastes has been experimentally evaluated. Davidson found that phenols could be chemically oxidized (90% complete) to carbon dioxide and water in ten minutes (9). Eisenhauer demonstrated that in dilute aqueous solutions of phenol, the reaction efficiency was considerably increased in the presence of air-available oxygen (7).

Eisenhauer also found that the reaction between a hydrogen peroxide-ferrous salt combination and ABS was rapid and 80 to 90% of the ABS was destroyed in the first ten minutes. Further ABS removals were obtained at slower reaction rates, with a 99% ABS removal effected after fifteen to twenty hours. The optimum reaction pH level was found to be from 3.0 to 3.5. The optimum concentrations of the reactants were six moles of ferrous salt per mole of ABS, and nine moles of H_2O_2 per mole of ABS. Multiple incremental additions of the ferrous salt will decrease the reaction time required (11).

Applying this information to a treatment of laundry wastes containing ABS, Eisenhauer obtained ABS reductions in excess of 90%, both with the raw waste and with effluent from a pretreatment process in which the detergent builders were precipitated. For a laundry waste containing 50 to 80 parts per million ABS, it was found that chemical costs ranged from $0.45 to $0.70 per 1,000 gallons for the raw waste treatment and $0.15 to $0.25 per 1,000 gallons for the effluent pretreated with ferric sulfate. This includes the cost of ferrous sulfate, ferric sulfate, sulfuric acid, and hydrogen peroxide (7).

Bishop et al (11) used hydrogen peroxide-iron salt systems to oxidize refractory organics in municipal wastewaters previously subjected to secondary biological treatment. Significant conclusions from this paper (11) are presented below.

1. The hydrogen peroxide-iron salt systems in various wastewaters oxidize a significant portion (≈70%) of the organics refractory to biological oxidation.
2. The oxidation process involves free radical ($\cdot OH$) oxidation and auto-oxidation and is effective only in the pH range of 3 to 4.
3. Oxidation efficiencies of 60% for the ferric ion systems and 30 to 51% for the ferrous systems were reported.

No data were found in the existing literature where hydrogen peroxide was used to oxidize raw sewage or combined sewer overflow. Since the H_2O_2 systems did oxidize the refractory organics present in sewage, there should be no problem oxidizing the organics in combined sewer overflow. The important problems associated with the use of this oxidation system for combined sewer overflow include:

1. The restricted pH range will require lowering of the pH of the combined sewer flow.
2. Materials used for construction will have to be able to withstand the pH of the system. This will require the use of stainless steel, or rubber coated mild carbon steel.
3. Removal of the ferric hydroxide formed during the reaction will require a flocculation and settling period.
4. The pH will have to be readjusted to near neutral before the effluent is discharged.

Ozone

The use of ozone as an oxidant for water and waste treatment is described in the literature (6)(13)(14)(15)(16)(17)(18)(19)(20)(21). Ozone is a blue gas under normal conditions. It contains three atoms of oxygen (O_3) and is heavier than air or oxygen. Ozone is one of the most powerful oxidants known (13). The oxidizing power of ozone is exceeded only by fluorine and compounds such as oxygen difluoride, atomic oxygen, and the hydroxyl radical (22). In high concentrations, ozone is toxic (13) (14). Figure I-1 shows the human tolerance for ozone at various concentrations and exposure times.

The area between the symptomatic line and irritant line of Figure I-1 is a nontoxic region. The threshold odor level of ozone is 0.01 to 0.02 ppm (V) which is well below the toxic region as shown in Figure I-1 (13). Exposed persons are thus given a warning of ozone's presence.

Source: Prolonged Ozone Inhalation 8, Its Effects on Visual Parameters, J.M. Langewerf, Aerospace Medicine, 36, June 1963

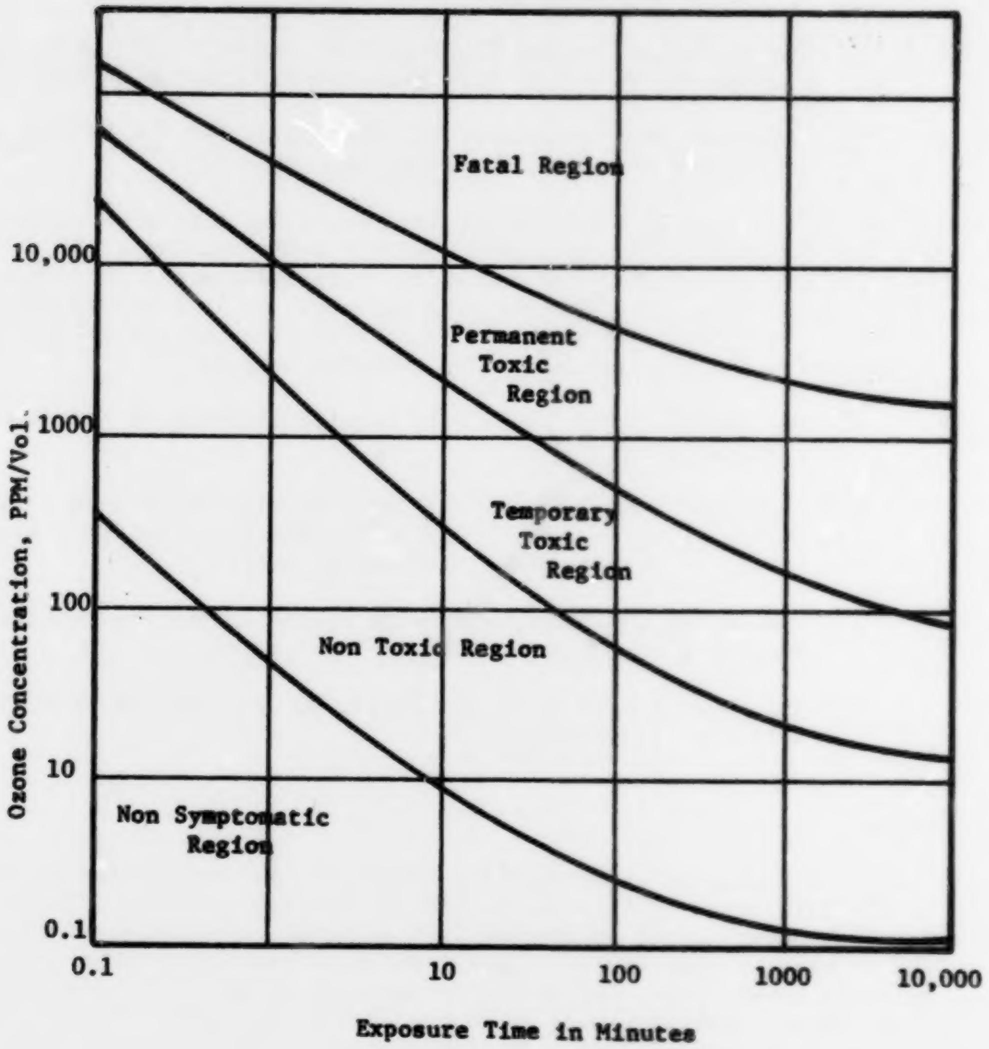


FIGURE I-1

HUMAN TOLERANCE FOR OZONE

Ozone can be produced in a number of ways. Among these are silent electric discharge, electrochemically, and chemically (22). Use of silent electric discharge is the principal upon which most laboratory and large scale ozonizers depend (13)(22). Production of ozone by this method requires the use of cold, clean air or oxygen. Basic equipment necessary include an air fan, equipment for filtering and drying the air and an ozonizer (14). When applying the ozone to water, some type of diffuser system and a contact tank will also be needed (14). Ozonizers produce ozone by passing the dried air between two concentric electrodes separated by a dielectric. High frequencies (500 cps to 1000 cps) and high voltages (4000 to 30,000 volts)are utilized. Some oxygen in the air passing through the ozonizer is broken into charged oxygen atoms, which recombine to form O_3. Ozonizers operating on air under pressure produce far greater quantities of ozone in higher concentrations than those operating in air at atmospheric pressures (13)(17). Commercial ozonators produce ozone concentrations of 1 to 4 percent by weight in air (17)(23). Power consumption is 9.5 to 11.5 KWH per pound of ozone produced (23).

Ozone is very corrosive and hence the materials used in the piping system carrying ozonated air must be carefully chosen. Porcelain has been used with success, as have aluminum, stainless steel, pressboard, oak, vinyl plastic and glass (14)(24). Polyvinyl chloride (PVC) has been used but there is a possibility that this material decomposes ozone (14).

The exact oxidation mechanism of ozone on organic materials is not thoroughly understood. Ozone decomposition may be a source of $\cdot OH$ radicals or the O_3 itself may be the actual oxidant (6)(21). Ozone is known, however, to be an effective oxidant (17)(18)(21). The exact oxidation products and intermediates resulting from ozone oxidation have not been defined. Evans and Ryckman (18) when ozonating secondary sewage treatment plant effluents have found that ozone readily oxidized ABS to intermediate compounds which were not detected by an ABS analysis. These intermediates actually caused an increase in BOD and COD values of the partially ozonated waste, and in fact destroyed the biological inertness of the ABS. Increased ozonation of this partially ozonated waste then caused a decrease in BOD and COD of the waste, apparently due to oxidation of some of the organic carbon to CO_2 and H_2O.

Andrews (16) was recently granted a patent on a new method of treating raw sewage. The method involves removal of the solids from raw sewage by a combination of settling, screening, and centrifuging. The resulting liquor, esentially free of solids and containing the soluble BOD fraction of the raw sewage, is ozonated to oxidize the BOD. The effluent from ozone treatment is filtered before discharging into the receiving stream.

The patent claims this method of treatment produces an effluent which is clear, colorless, free from bacteria and contains a high quantity of dissolved oxygen. No reference is made, however, as to dosages of ozone required and BOD content of the effluent.

Miller (21) reports use of ozone to control sewer odors at sewage lift stations. He states ozone addition reduced the raw sewage BOD from

120 to 32-39 mg/l. Septic conditions in flat sewers disappeared and the plant influent contained 2 to 4 mg/l of dissolved oxygen. No mention was made of the quantities of ozone added. The ozone was injected directly into the pump wet wells for conventional lift stations and into the air compressors supplying air for the pneumatic-ejector type lift stations.

March and Panula (15) reported on a study where raw sewage was ozonated. They report that the BOD of the sewage which was ozonated decreased from 400 to 300 mg/l. Application rate was 6 mg/l of ozone actually absorbed by the sewage. This indicates that 1 mg of O_3 will oxidize about 16 mg of BOD which seems quite high. The amount of O_3 applied during this study was greater than the amount actually utilized. Absorption efficiencies averaged 22.6%. Hence, the actual ozone dosage was about 26 mg/l. March and Panula (15) indicate that the biggest problem in full-scale ozone treatment is efficient use of the ozone being produced. They suggest collecting the unabsorbed ozone and reapplying it to the sewage to obtain better absorption efficiencies. It should be noted that a simple diffusion system was used during this study with a 3½ foot depth of liquid over the diffusers. Bubble size was on the order of 0.01 inches in diameter.

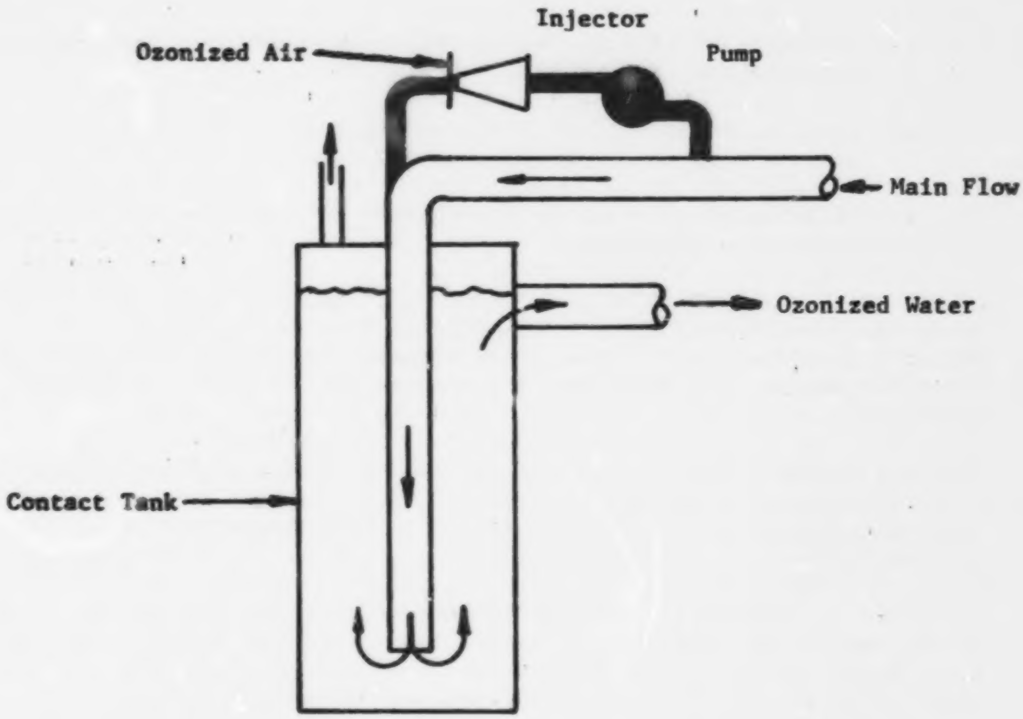
O'Donovan (14) stresses the importance of mixing techniques when introducing ozone into water. He stresses small bubble diameter and turbulence to be important in obtaining good O_3 adsorption efficiencies. Two types of adsorption systems which have been used are illustrated in Figure I-2 (14)(25). Campbell (25) reports 90% utilization of ozone when using a partial-ejection system.

The use of ozone to oxidize combined sewer overflow appears to hold considerable promise. Ozone has many of the characteristics of the "ideal oxidant" as previously discussed. It is apparently nonspecific in its oxidation of organic material (21), the necessary contact time is less than 30 minutes (14)(25), it oxidizes at the prevailing pH of the waste (25), and is not difficult to dispense(14). Some disadvantages involved with the use of ozone include precipitation of iron and manganese when present in concentrations greater than 0.2 mg/l Fe and 0.1 mg/l Mn, and inability to store ozone requires ozonators at each application site. Recently, however, Matheson Inc. has announced it has a process which will allow storage of ozone which could eliminate this latter disadvantage.

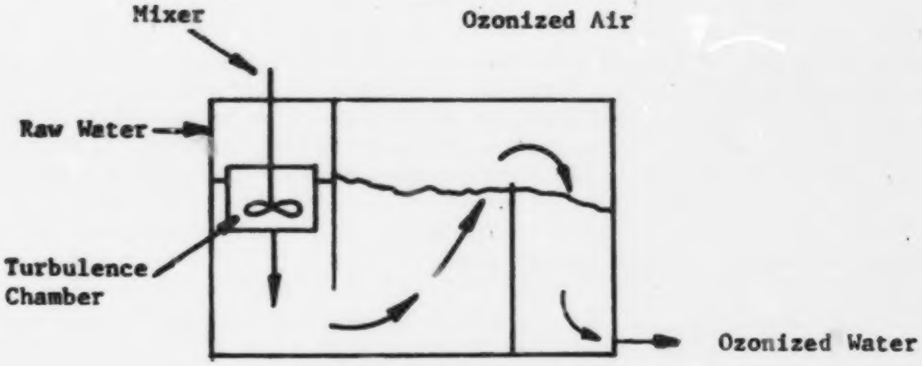
Oxidation by Chlorine

Chlorine or its derivatives are added to most municipal water supplied in the United States to kill micro-organisms. Green and Stumpf (26) have shown that chlorine compounds react with certain enzymes which are essential to the metabolic process of living cells and that death results from the inactivation of these enzymes. Dosages required for disinfection are low, 0.2 to 2 mg/l (1). At these dosages, probably little, if any, oxidation of organic material is occurring.

Use of chlorine to reduce the BOD of sewage has been practiced. Four kinds of reactions could be involved in reducing BOD by chlorination (1)(27).



Partial-Injection
(A)



Kerag System
(B)

FIGURE I-2

OZONE ABSORPTION SYSTEMS

1. Direct oxidation of organic materials.

2. Substitution of chlorine for hydrogen which produces compounds which have a bactericidal power.

3. Substitution of chlorine for hydrogen which produces compounds which are no longer biologically degradable.

4. Addition of chlorine to unsaturated compounds forming nonbiodegradable substances.

Griffin and Chamberlin (28) have shown that chlorination of sewage does result in a reduction of BOD. After an 18 hour contact period, about 35% of the BOD had been removed from screened raw sewage at a 200 mg/l chlorine dosage. Increasing the dosage had little effect on further removals.

Although chlorine and its derivatives do oxidize many of the organic chemicals contained in municipal wastewaters, they do not usually convert them to the readily acceptable forms of carbon dioxide and water (6). Instead, the reaction of chlorine with organics may produce molecules having considerable taste and odor, and complex compounds which serve as secondary pollutants may also be formed. The addition of stoichiometric amounts of chlorine will not effect oxidation of the organics, and a large chlorine residual may remain in the treated waste (6)(28). Since oxidation by chlorine is far less effective than by active-oxygen elements and since the unknown by-products of chlorine oxidation may be toxic, chlorine and its derivatives are not considered to be good prospects for removal of large amounts of organic pollutants from wastewaters by oxidation (6).

A method of making chlorine more reactive could, however, make the use of chlorine more attractive. Use of ultraviolet radiation to catalyze the oxidation of organic material by chlorine has been suggested (29). No specific data on such a process was found in the literature, but the process will be investigated during the laboratory phase of this study.

Oxidation by Oxy-Acids and Their Salts

The oxy-acid oxidants are usually added to aqueous systems as the salt of the oxy-acid. Important oxy-acid salts with high oxidation potentials include potassium permanganate ($KMNO_4$), sodium ferrate ($NaFeO_4$) and potassium ferrate (K_2FeO_4). The oxidation potentials of the oxy-acids are strongly dependent upon the acidity of the aqueous system, with the potential increasing as the acidity increases (6).

Potassium permanganate ($KMNO_4$) is used as an oxidant to disinfect water supplies (1)(30) and as an oxidizing agent in the permangate oxygen demand test (31). However, references in the literature report only limited oxidation of organics with $KMNO_4$, and a residual permanganate persists in the water for more than twenty-four hours (30)(31). Vosloo

(32) reports that the permanganate concentration necessary for maximum oxidation should be about twice the concentration of permanganate which will be utilized. The use of potassium permanganate would therefore require removal of the excess MNO_4-ion as a part of the treatment process, and this would greatly increase the salt load in the aqueous system (6). The potential for using $KMNO_4$ as an oxidant in this project is not at all promising, and hence, $KMNO_4$ oxidation will not be considered.

Although ferrate salts such as potassium ferrate (K_2FeO_4) are not commercially produced, their high oxidation potentials (33) and the coagulating properties of their oxidation products, a highly insoluble ferric hydroxide, make the ferrates a potentially attractive group of oxidants for treatment of wastewaters (6).

Exploratory laboratory tests using K_2FeO_4 to treat filtered municipal secondary effluents in dosages supplying approximately 100 milligrams per liter of available oxygen showed the combination of ferrate oxidation and coagulation to be nearly as effective as the previously discussed H_2O_2 - iron salt system for removing organic materials from wastewaters (6). The ferrate salts, however, also possess the same disadvantages as the H_2O_2 iron salt system, (see previous discussion on hydrogen peroxide oxidation) and these disadvantages must be considered when attempting to use the ferrate salts as oxidants in this project.

Electro-Chemical Oxidation

Any material that increases the electrical conductivity of an aqueous solution may enter into a chemical reaction at the surface of electrodes placed in the solution (27). Interest in electrochemistry as a possible technique for purifying municipal wastewaters is based on the fact that many organic chemicals take part in such electrode reactions, often resulting in the complete degradation of complex organic molecules to carbon dioxide, water and other oxides. After reviewing a recent study on electrochemical oxidation (34), it was concluded that electrochemical oxidation did not hold potential for treating combined sewer overflows for the following reasons:

1. High operating and maintenance costs.
2. Formation of precipitates which would have to be removed from the waste flow.
3. Long reaction times are necessary --2 to 5 hours.

Combination of Oxidants

Although no specific data were found in the literature about combining various oxidants, a combination of ozone and H_2O_2 or other oxidant combinations may produce an effective oxidation system. For this reason, various combinations of oxidants will be evaluated during the laboratory phase of this project.

LABORATORY INVESTIGATIONS

General Test Procedures

The literature search indicated that the oxidants with the highest potential success were ozone, hydrogen peroxide, and chlorine. During the laboratory investigation phase the above mentioned oxidants as well as various oxidant combinations were evaluated.

Samples of seven separate combined sewer overflows from the demonstration site at Hawley Road were collected during the Fall of 1967. These samples were analyzed in the laboratory within twenty-four hours of their collection for BOD, COD, dissolved COD, total solids, suspended solids, volatile suspended solids, coliform density and pH. The data from these analyses has been reported and discussed in the main body of this report. Since the likelihood of getting sufficient and frequent combined overflows during the fall and winter seasons in Milwaukee is small, it was decided to freeze some combined overflow samples and use them for chemical oxidation evaluation during dry weather. Therefore, about twenty liters of combined overflow samples from three separate storms was frozen in individual containers having volumes of from 100 to 200 milliliters. The laboratory analysis of the three storm samples frozen for chemical oxidation studies is shown in Table I-1. During dry periods, samples were thawed as needed. Samples were mixed thoroughly in a Waring blinder after thawing to provide a colloidal system characteristic of the original unfrozen raw sample. Whenever a sample without the particulate matter was needed for oxidation of the dissolved organic matter, the thawed sample was not mixed. Instead, the supernatant was filtered through a SS-597 filter paper and millipore filter discs to obtain a dissolved sample. Standard Method (35) procedures were employed for various analysis in the laboratory and are discussed in Appendix II.

Special Test Procedures with Various Chemical Oxidants

Chlorine and Hydrogen peroxide oxidations were performed with and without ultra-violet (UV) light. Chlorine was applied in the form of calcium hypochloride solution (HTH) and a 0.75% stock solution was utilized for hydrogen peroxide oxidation. Procedures for determination of oxidant dosages and residuals are discussed in Appendix II. Cobalt in the form of $CoCl_2 \cdot 6\ H_2O$ was utilized in a number of tests to determine if it helped to catalyze oxidation reactions of H_2O_2 and chlorine. The apparatus consisted of an eight inch diameter flat bottomed dish on a magnetic stirrer under a protective hood cover. A 300 ml aliquot of raw waste was used. For tests performed with ultra-violet light, the UV lamp was suspended above the water surface with ring stands at three and six inch heights. The two UV lamps utilized were a single bulb Sperti sunlamp and a double bulb G.E. germicidal lamp. Figure I-3 shows a photograph of the apparatus utilized for these tests. Samples were taken at various intervals during the oxidation period for COD analysis.

TABLE I-1

LABORATORY ANALYSIS OF COMBINED SEWER SAMPLES UTILIZED FOR CHEMICAL OXIDATION STUDY

Date	pH	Total Solids mg/1	Suspended Solids mg/1	Volatile Suspended Solids mg/1	COD mg/1	Dissolved COD mg/1	BOD mg/1	Coliforms per ml
11/25/67	7.4	649	138	--	159	--	59	4260
12/07/67	6.6	--	232	113	298	50	--	2151
01/29/68	7.1	--	2158	--	1410	60	--	5730

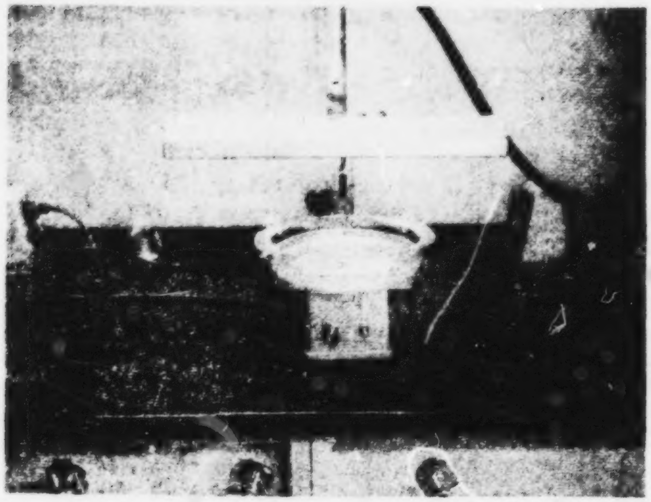


FIGURE I-3

APPARATUS FOR ULTRA VIOLET LIGHT OXIDATIONS

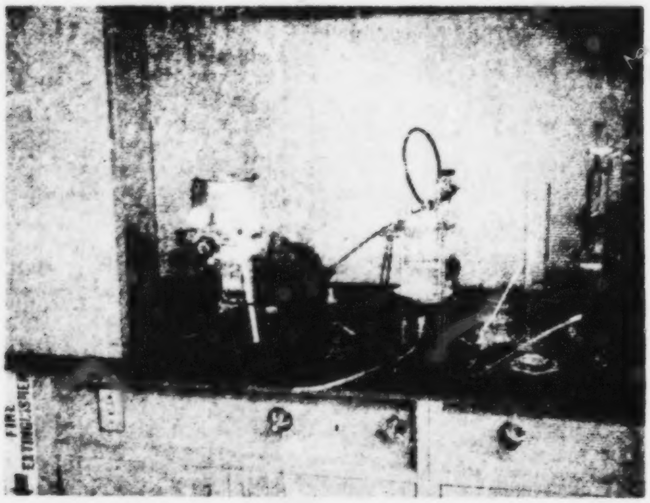


FIGURE I-4

APPARATUS FOR OZONE OXIDATIONS

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Some difficulty was encountered in measuring the COD after oxidation with various oxidants. Contrary to expectations, some samples exhibited an increase in COD after oxidation. This observation could have been caused by the break down of organic material which prior to oxidation was not measured by the COD tests. The increase may also have been caused by the excess oxidant remaining (the reaction kinetics were not known and exact amounts of oxidants could not be utilized). Since it was desirable to know if the COD was increasing due to partial oxidation, the excess oxidants were removed prior to analyzing for COD. Several trial and error procedures were investigated for the removal of these excess oxidants. It was planned that excess oxidant would be removed after oxidation tests by adding a small excess of sodium sulfite. After the addition of sodium sulfite, the sample was aerated until a galvanic cell oxygen analyzer indicated the presence of dissolved oxygen in the sample. Also investigations were undertaken to study the effect of excess oxidants and chemicals on the COD test results. Theoretically, the reaction between hydrogen peroxide and hexavalent chromium in a COD test may proceed as follows:

$3H_2O_2 + K_2Cr_2O_7 + 8H^+ = 3O_2 + 2Cr^{+3} + 7H_2O_2 + 2K^+$ [ORP = +0.65].... EQ I

Then 1 mg H_2O_2 = 0.47 mg COD

Similarly, the reaction between sodium sulfite and dichromate may proceed as follows:

$3Na_2SO_3 + K_2Cr_2O_7 + 8H^+ = 3Na_2SO_4 + 2Cr^{+3} + 4H_2O + 2K^+$ ..... EQ II

Then 1 mg Na_2SO_3 = 0.127 mg/COD

Experimentally it was found that the COD exerted by hydrogen peroxide solution decreased with increasing strength of H_2O_2 solution as follows:

Concentration of H_2O_2 Solution	Ratio of mg COD/mg H_2O_2
50	0.42
100	0.41
150	0.39
200	0.38
250	0.37

The reason for such observation can be attributed to the redox relationships of hydrogen peroxide, since H_2O_2 can also react with the trivalent chromium produced as follows:

$$3H_2O_2 + 2Cr^{+3} + 6H^{+} + 7H_2O = 6H_2O + Cr_2O_7^{=} + 14H^{+} \text{ [ORP = +.44]..... EQ III}$$

Thus hydrogen peroxide solution was found to exert a significant COD value and this was a factor in the increased COD results of hydrogen peroxide treated samples.

Also, sodium sulfite was experimentally found to have a chemical oxygen demand of 0.09 mg per mg of Na_2SO_3. This is approximately 70% of the theoretically calculated value of 0.127 and is not significant in light of the small amounts of sodium sulfite utilized in these studies.

When calcium hypochlorite was used as an oxidant, the sulfite was added in increments until a spot test with ortho-tolidine indicated that there was no more free chlorine. The sample was then aerated until a galvanic cell oxygen analyzer indicated the presence of dissolved oxygen in the sample. Experimentally it had been found that a 715 mg/l calcium hypochlorite solution exerted a nominal COD of 9 mg/l which could possibly be the result of experimental errors. This experimental observation is also supported by possible theoretical reaction between chlorine and hexavalent chromium, as:

$$3Cl_2 + 5Cr_2O_7^{=} + 34H^{+} = 6ClO_3^{-} + 10Cr^{+3} + 17H_2O \text{ [ORP = -.14].... Eq IV}$$

The negative oxidation reduction potential indicates that the above reaction should proceed to the left and therefore, theoretically the reaction should not take place. This leads to the conclusion that no COD should be exerted by the chlorine solution.

Somewhat contradictory results were obtained when COD tests were performed on samples from stormwater oxidation tests, whereas the COD of calcium hypochlorite solution was negligible and the COD of oxidized sodium sulfite was very small, in most cases the COD values of chlorinated waste samples which were dechlorinated with sodium sulfite and aerated were 5 to 35% higher than the samples which were not dechlorinated. Because of this difference, an alternate method of dechlorination was also used: the sample was acidified to a pH of one and stripped of chlorine by aeration. Unfortunately, the stripping process was extremely slow, often requiring several hours. In some cases, not all the chlorine was removed by the time the COD analyses were performed. In general, the COD values obtained by acidification and stripping were lower than the values obtained by sulfite treatment. The exact cause of this difference in values is not known.

Ozone Oxidation System

Ozone was purchased by the bottle and used in a pressure tank system. A photograph of the apparatus utilized is shown in Figure I-4 and a schematic is shown in Figure I-5. Ozone was supplied dissolved in 'Freon 13' under high pressure in stainless steel cylinders. Since ozone has a half life of approximately three days at room temperature it was necessary to keep the ozone cylinder packed in dry ice. A three mole-percent vapor phase concentration was utilized for oxidation studies. To maintain a constant ozone concentration supply, ozone was utilized in the vapor phase. This was achieved by inverting the ozone bottle and converting the liquid ozone to vapor phase by passing it through a vaporizing coil.

Ozone was mixed with the waste at elevated pressures in the range of 40 to 80 psig. This pressure charged stream was then placed in a graduated cylinder for the required reaction time. Samples were air-stripped to remove excess ozone remaining after the reaction time. Pressurization of waste could be total or partial. When the entire waste was pressurized, it was called total pressurization. If only a portion of the waste was pressurized, the amount was calculated as a percentage of the nonpressurized portion, and was called a recycle system. Use of a pressure tank system enabled large amounts of ozone to be dissolved and when the pressure was released the extremely small air-ozone bubbles stayed in contact with the liquid much longer than the coarse bubbles produced from diffuser type systems. Such a system was utilized because the literature search indicated that the ozone contact time is critical and greatly affects efficient use of ozone. Titrations for ozone determination were performed in accordance with Standard Methods(35).

RESULTS AND DISCUSSION

Oxidation with Hydrogen Peroxide

Results of chemical oxidation with hydrogen peroxide are presented in Tables I-2 and I-3. Hydrogen peroxide requires a catalyst to oxidize effectively within reaction times of one hour or less. Table I-2 shows the results of the tests performed using iron as a catalyst. Oxidation tests were run on the dissolved fraction of the overflow

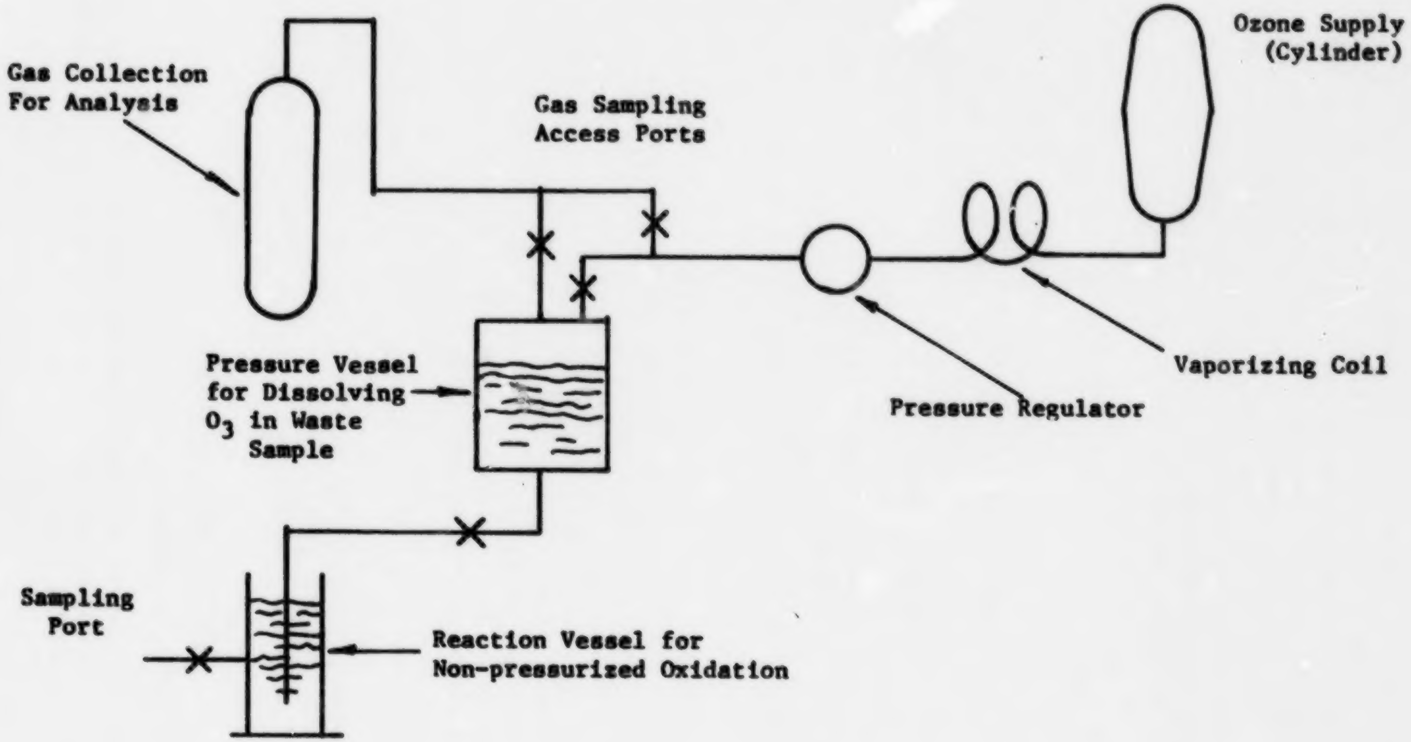

FIGURE I-5

SCHEMATIC OF OZONE TEST APPARATUS

TABLE I-2

RESULTS OF CHEMICAL OXIDATION OF COMBINED SEWER OVERFLOW WITH HYDROGEN PEROXIDE

Test No.	Available O_2	COD Initial	COD Final	Percent Reduction	Oxidation Efficiency Percent	Filtered Prior to Analysis
1	50	54	50	7	8	No
2	100	67	30	55	37	Yes
3	100	54	41	24	13	No
4	100	54	33	39	21	Yes
5	100	65	37	43	28	Yes
6(3)	100	65	42	35	23	Yes
7(4)	100	65	31	52	34	Yes
8	150	216	186	14	20	No
9	300	216	185	14	10	No

NOTES:

1. In Tests 1 through 7 only the dissolved COD was oxidized, i.e, all solids removed prior to oxidation. Tests 8 and 9 were run on combined overflow after screening through a 50 mesh screen.
2. All oxidations were for 15 minutes at room temperature.
3. Aerated during the oxidation.
4. Cobalt added during the oxidation.

TABLE I-3

EFFECT OF UV LIGHT AND COBALT ON HYDROGEN PEROXIDE OXIDATION

Test No.	Type of Sample	H_2O_2 Dosage mg/l as O_2	H_2O_2 Remaining after Oxidation mg/l as O_2	Catalyst Used	Reaction Time	COD, mg/l Influent	COD, mg/l After Oxidation	% COD Reduction
1	Filtered	110	--	None	15	92	62	33.0
					30		75	19.0
					90		175	--
2	Filtered	110	--	UV Light[1]	15	92	149	--
					30		161	--
					90		138	--
3	Filtered	110	29	UV Light & 10 mg/l Cobalt	30	106	100	5.6
4	Filtered	110	15	UV Light & 100 mg/l Cobalt	30	106	98	7.5
5	Filtered	110	66	10 mg/l Cobalt	30	106	119	--
6	Filtered	110	54	100 mg/l Cobalt	30	106	114	--
7	Filtered	110	60	UV Light	30	58	34	41.0
8	Filtered	110	62	UV Light &	30	58	39	33.0
9	Distilled Water	110	93	UV Light	30	--	80	--
10	Distilled Water	110	106	UV Light & 5 mg/l Cobalt	30	--	81	--
11	Filtered	110	78	UV Light	15	114	76	34.0
12	Filtered	110	66	UV Light	30	114	107	6.0
13	Filtered	110	79	UV Light	15	112	59	47.0
14	Filtered	110	68	UV Light	30	112	80	29.0

NOTES:

1. Ultra-violet light wave length 2800 to 3200 Å.

(Tests 1 through 7) as well as the overflow after screening through a 50 mesh screen (Tests 8 and 9).

Test 1 through 7 (Table I-2) indicated that a substantial portion of the dissolved COD can be oxidized in 15 minutes and the oxidation efficiencies were in the range of those reported in the literature. When the effluent from the 50 mesh screen (Tests 8 and 9) was oxidized with the hydrogen peroxide - iron salt system, the oxidation efficiencies were lower than those obtained when oxidizing only dissolved COD. This indicated the importance of removing a major portion of the solids prior to oxidation, since particulate matter is more difficult to oxidize than dissolved organic matter.

Use of hydrogen peroxide-iron salt system does not appear feasible. Ferrous iron seems to work well, but relatively large concentrations are required (on the order of 60 to 100 mg/l as Fe^{++}). This system requires strict pH control (3 to 4) and a neutralization and settling period to remove the iron precipitate which is formed. Organic removal efficiencies are in the 30 to 50% range for dissolved organics and were reduced to about 14% when particulate organics were present. In an attempt to more efficiently utilize the hydrogen peroxide, other catalysts were evaluated which would not be as restrictive as ferrous salts.

The literature indicated that hydrogen peroxide is decomposed by ultra-violet light. Also Test 7 (Table I-2) indicated that cobalt may help catalyze a hydrogen peroxide system. Therefore, tests were run with both these catalysts to study any improvements in the efficiency of hydrogen peroxide oxidation system. The results of these tests are shown in Table I-3. It is seen that the use of either or both cobalt and ultra-violet light along with hydrogen peroxide did not produce consistent results. From Tests 1 and 2 (Table I-3) it is clear that reaction time has an important bearing on COD reductions in the treatment of storm waters with hydrogen peroxide. In Test 1, when a hydrogen peroxide dosage of 110 mg/l as O_2 was utilized, a 33% reduction in COD was obtained in 15 minutes; the COD reduction reduced to 19% in 30 minutes and even became negative at the end of the 90 minute reaction time. A possible explanation for the increase in COD in a certain initial reaction time may be the partial oxidation of large molecular weight organic compounds in the raw storm samples to smaller intermediate compounds which exhibited an increase in COD. Increased oxidation at a higher reaction time may then cause a decrease in COD of these samples, apparently, due to the oxidation of some of the organic carbon to CO_2 and H_2O. Similar explanations have been mentioned in literature for ozone oxidation(18). When UV light was used as a catalyst, results were contradictory as shown in Tests 2, 7, 11, 12, 13 and 14 (Table I-3). Test 2 showed negative COD reductions for all reaction times between 15 to 90 minutes while Tests 7 and 11 to 14 showed COD reduction between 6 to 47% for reaction times of 15 and 30 minutes. Similar contradictions in results were shown when cobalt was used as a catalyst either independently or in combination with UV light (Tests 3 to 6, Table I-3).

The results presented in Tables I-2 and I-3 clearly indicated that a Hydrogen Peroxide Reaction System was not technically feasible for treating combined sewer overflows. Thus work on such a system was abandoned.

Oxidation with Chlorine

Results of chlorine oxidation with and without ultra-violet light are presented in Tables I-4 and I-5. Oxidation with chlorine and no ultra-violet light resulted in a reduction of COD of about 20 to 25% at available chlorine dosages of 280 to 560 mg/l (Table I-4). The amount of chlorine destroyed was about 25% and hence a large chlorine residual remained after the oxidation. Reducing the chlorine dosage of 56 mg/l, reduced the percent COD oxidized to only 8% (Test 3, Table I-4). These results indicate that chlorine is not an effective oxidant for combined sewer overflows. This conclusion is supported by the literature search.

Oxidation with chlorine and ultra-violet light in the 2800 to 3200 Å range was not significantly different from those oxidations where ultra-violet light was not used (Table I-4). The reason for these low oxidation efficiencies may have been due to the long wave length light which was used i.e. 2800 to 3200°A or the low ultra-violet output of the lamp which was used for the reactions. A review of the research conducted by Midwest Research Institute (36) indicated that lower wave lengths were more effective. Therefore, a lower UV light wave length of 2587 Å was used for further experiments. This wave length would also provide good disinfection. The results of these experiments are shown in Table I-5. It should be noted that for light catalyzed oxidations to be effective, relatively clear solutions were required. Therefore, the results of the screening step could have a pronounced influence on the efficiency of light catalyzed oxidations.

From the results shown in Table I-5, it is seen that COD reductions of 10% to 50% can be expected depending upon light intensity, chlorine dosage and reaction time. The amount of chlorine required is extremely large. Approximately 7 to 10 mg of chlorine is required per mg of COD oxidized. Hence, for a 200 mg/l COD waste, 1400 to 2000 mg/l of chlorine would be consumed. This also results in high chlorine residuals in the effluent. With the use of an ultra-violet light catalyst, organic removal does increase to the 25 to 50% level compared to 10-25% level without the catalyst, but large chlorine residuals are still present in the reactor effluent. This fact, coupled with relatively long reaction times (30 to 60 minutes) and poor light penetration when the waste is turbid appear to eliminate the possibility of using light catalyzed chlorine to oxidize combined sewer overflows.

Oxidation with Combined System of Hydrogen Peroxide and Chlorine

Results of oxidation with the combined system of hydrogen peroxide and chlorine with and without ultra-violet light catalyst are shown in Table I-6. The results of these tests were extremely discouraging as

TABLE I-4

CHLORINE OXIDATION TESTS

Test No.	Chlorine Concentration mg/1	Reaction Time Min.	COD mg/1			% COD Reduction		Chlorine Remaining	
				After Oxidation					
			Raw	W/UV	No Light	W/UV[2]	No UV	W/UV	No UV
1[1]	280	15	65	---	51	---	21	---	
2	560	15	259	211	194	19	25	---	
3	56	15	177	167	163	6	8	---	
4	276	15	198	169	164	15	17	160	202
5	284	15	205	151	161	26	21	145	216

NOTES:
1. Sample filtered prior to oxidation
2. Ultra-violet wave length 2800 to 3200 Å.
3. Ca $(OCl)_2$ used as source of chlorine.

TABLE I-5

LIGHT CATALYZED CHLORINE OXIDATION

Date 1969	Type of Sample	Reaction Time Min.	COD mg/1 Influent	COD mg/1 After Oxidation	Percent COD Reduction	Chlorine Dosage mg/1	Chlorine Remaining mg/1	U.V. Light Intensity Watts[1]	Chloride Conc. mg/1	Cl/COD Ratio 2
2/2	Filtered	15	105	94	11	280	115	2.9	---	15
2/2	Filtered	30	105	86	18	280	70	2.9	---	11
2/2	Filtered	60	105	77	27	280	42	2.9	---	8.5
2/2	Filtered	15	105	91	13	280	185	0	---	6.
2/2	Filtered	30	105	89	15	280	182	0	---	6.1
2/2	Filtered	60	105	96	9	280	182	0	---	10.9
2/8	Filtered	15	71	43	39	600	401	2.9	---	7.1
2/8	Filtered	30	71	44	38	600	309	2.9	---	10.8
2/8	Filtered	60	71	37	48	600	204	2.9	---	11.6
2/9	Raw	15	147	121	18	600	384	2.9	45	8.3
2/9	Raw	30	147	111	25	600	283	2.9	82	8.8
2/9	Raw	60	147	102	31	600	286	2.9	253	7.0
2/13	Filtered	15	97	81	17	600	497	0	---	6.6
2/13	Filtered	30	97	77	21	600	497	0	---	5.7
2/13	Filtered	60	97	77	21	600	497	0	37	5.7
2/16	Filtered	30	108	76	30	900	576	2.9	382	10.
2/16	Filtered	30	108	60	44	900	541	5.8	502	7.5
2/16	Raw	30	167	118	29	900	576	2.9	397	6.6
2/16	Raw	30	167	68	59	900	497	5.8	487	4.1
2/26	Filtered	30	93	72	23	300	84	5.8	352	10.3

NOTES: 1) U.V. Light Wave Length 2587 Å - 2.9 watts intensity (bulb output).
2) Ratio is mg Cl utilized per mg COD oxidized.

TABLE I-6

OXIDATION WITH COMBINED SYSTEM OF HYDROGEN PEROXIDE AND CHLORINE

Test No.	Type of Sample	Chlorine Dosage mg/1	H_2O_2 Dosage mg/1 as O_2	Catalyst Used	Reaction Time	COD, mg/1 Influent	COD, mg/1 After[1] Oxidation	% COD Reduction
1	Filtered[2]	110	100	None	15	93	81	13.0
					30		95	---
					60		122	---
2	Filtered	110	100	UV Light[2]	15	93	115	---
					30		117	---
					60		127	
3	Filtered	110	100	UV Light	Instant	86	80	7.0
					15		104	---
					30		108	---
					45		104	---
					60		85	
4	Filtered	110	100	UV Light	Instant	86	82	4.8
					15		99	---
					30		106	---
					45		100	---
					60		113	---

NOTES:

1. After stripping of the excess oxidant .
2. Ultra-violet light wave length 2587 A.

inconsistent and negative reductions in COD were observed. The importance of reaction time was again demonstrated (Tests 1 to 4, Table I-6) in the use of the combination of these oxidants for the treatment of storm wastes. These results were similar to the ones discussed earlier for oxidation by hydrogen peroxide alone. A small positive COD reduction was shown when reaction time was very small (instantaneous sample, Tests 3 and 4, Table I-6). For higher reaction times up to 60 minutes negative COD reductions were obtained. Also the use of U.V. light as a catalyst did not show any improvement in results (Tests 2, 3 and 4 compared to 1, Table I-6). Hence, investigations in this area were terminated.

Oxidation with Ozone

The result of the ozone oxidations are shown in Table I-7. Using total pressurization at 40 psig Tests 1 and 2 show the difference in COD reductions for raw and filtered storm water samples. The higher COD reduction of 46% exhibited for raw waste sample (Test 2, Table I-7) was probably due to the flotation of some of the suspended matter as well as the higher solids content of the raw waste sample. When a 20% recycle system was used with the 10 mg/l dosage of polymer C-31 on raw sample (Test 4) the COD reduction was extremely high at 96%. The reason for such a high reduction was probably the flocculation of the suspended matter with the C-31 flocculant and the flotation of floc on the surface. Tests 5 and 6 (Table I-7) show the effect of reaction time of ozone at two different operating pressures of 40 and 70 psig. It was shown that COD reductions increased from 27 to 52% when the reaction time was increased from 1 to 20 minutes at 40 psig. There was no significant improvement in COD reductions when the operating pressure was increased to 70 psig. Therefore, a pressure of 40 psig was utilized for all future tests. The ozone dosage utilized for all the tests discussed above was 40 mg/l. When a 30 mg/l ozone dosage was utilized the COD reduction was 32% (Test I-7) for a reaction time of five minutes as compared to 43% COD reduction shown for 40 mg/l for corresponding conditions (Test I-5). This means that higher ozone dosages result in improved COD reductions. When ozone was applied in combination with other oxidants (such as Cl_2 and H_2O_2) or catalysts (such as Co and Ni) (Tests 8 through 18), no significant improvement in COD reductions could be obtained in most cases either with dissolved or raw storm water samples. However, in one case where ozone was used with both chlorine and hydrogen peroxide (Test 10), the COD reductions were observed to be significantly improved at 70%. However, this efficiency was only for the dissolved organic fraction and would be reduced significantly if particulate organic materials are present. The use of all three oxidants at their respective dosages (Test 10, Table I-7) would result in extremely high(>$1.00/1000 gal) operating costs.

Two benefits which are possible when using a pressure tank system are the possibility of floating some of the lighter organic solids, and the probability that the heavier grit particles will settle. These benefits can greatly increase the removal efficiencies of the treatment system. From the results shown in Table I-7, ozone appears to be the best oxidant among the ones evaluated in this study because it:

TABLE I-7

SUMMARY O_3 OXIDATION TESTS

Test No.	Type Sample	Reaction Time Min.	Raw	After Oxidation	% COD Removed	Type Recycle	O_3 Dissolving Pressure psig	Approx. O_3 Dosage mg/l	Remarks
1	Filtered	3	74	55	26	Total	40	40	Excess O_3
2	Raw	3	1680	959	43	Total	40	--	No Excess O_3
3	Filtered	3	73	49	33	Total	40	40	Excess O_3
4	Raw	3	1680	61	96	20%	40	--	10 mg/l C-31
5	Filtered	1	69	50	27	Total	40	40	Excess O_3
5	Filtered	5	69	43	38	Total	40	40	Excess O_3
5	Filtered	10	69	40	42	Total	40	40	Excess O_3
5	Filtered	20	69	33	52	Total	40	40	Excess O_3
6	Filtered	1	69	51	26	Total	70	--	Excess O_3
6	Filtered	5	69	42	39	Total	70	--	Excess O_3
6	Filtered	10	69	36	48	Total	70	--	Excess O_3
6	Filtered	20	69	31	55	Total	70	--	Excess O_3
7	Filtered	5	69	46	32	Total	40	30	Excess O_3
8	Filtered	5	69	46	33	Total	40	30	50 mg/l H_2O_2
9	Filtered	5	69	45	35	Total	40	30	50 mg/l Cl
10	Filtered	5	69	21	70	Total	40	30	50 mg/l Cl and H_2O_2
11	Filtered	8	84	61	27	Total	[illegible]	30	
12	Filtered	8	84	62	26	Total	40	30	5 mg/l Co added
13	Raw	8	189	92	51	Total	40	--	
14	Raw	8	189	154	19	20%	40	10	
15	Raw	8	189	114	40	Total	40	--	25 mg/l H_2O_2 and Cl
16	Raw	8	189	120	37	Total	40	36	Actual measurement
17	Raw	8	189	1414	25	20%	40	8	
18	Raw	8	189	116	39	Total	40	67	3 mg/l Ni added actual

NOTE: Samples collected fall of 1967 and frozen for subsequent use.

1. Provides the best oxidation
2. Requires reasonable reaction time
3. Can be used in a pressure system allowing effective introduction of ozone

It should be noted that all the tests shown in Table I-7 were obtained with overflow samples which had earlier been collected and frozen. Therefore, it was not possible to run disinfection tests because of the adverse effect that freezing has on the coliform organisms. Hence, further experiments were planned to study disinfection efficiency and BOD and COD removals using ozone treatment and fresh overflows.

The results of these experiments are shown in Tables I-8 and I-9. A recycle system was utilized and a value of 10% recycle was used to keep the ozone dosage in 10 to 40 mg/l range on all overflows except the April 3 overflow (Tables I-8 and I-9). Since some flotation and/or sedimentation occurs, air flotation tests were run as a control to determine how much removal was due to oxidation and how much to sedimentation and/or flotation.

Comparing the suspended solids removals (Table I-8) indicates little difference between ozone treatment and air flotation. This points out that little if any particulate matter is oxidized. These results are consistent with the oxidation results obtained with the frozen overflows. The BOD removal was generally better with the ozone treatment compared to air flotation (Table I-8). The COD removals, however, followed the opposite trend. Overall removal rates with screening and ozone treatment or flotation are in the range of 50 to 80% for the first flushes and 45-65% for the normal overflows providing means are available to remove settled and floated solids.

The amount of disinfection obtained with ozone and chlorine is shown in Table I-9. At relatively high ozone dosages good disinfection is obtained. When the dosage is dropped below 10 mg/l disinfection becomes very poor. Chlorine, however, will give good disinfection at a 10 mg/l dosage over a wide range of coliform concentrations. These results indicate that ozone may not be satisfactory for disinfection because of its high costs and nonattainment of residual ozone for a sufficient period.

Oxidation With Gamma Radiation

Use of gamma radiation for oxidizing organics was indicated in the literature (38)(39). Tests were run on gamma irradiation of combined overflow using a cobalt source. The results of preliminary tests on these were discouraging and investigations into this area were terminated.

TABLE I-8

SUMMARY O_3 OXIDATION AND AIR FLOTATION TESTS

Date	Type	SS After Screening	SS After Oxidation	SS After Air Flotation	BOD After Screening	BOD After Oxidation	BOD After Air Flotation	COD After Screening	COD After Oxidation	COD Air Flotation
4/3	FF	707	279	---	148	85	---	559	---	---
4/3	EO	183	114	---	33	12	---	121	---	---
4/17	EO	207	106	73	33	17	21	134	79	69
4/20	EO	67	48	55	13	10	10	51	36	44
4/23	EO	122	86	65	21	20	12	99	83	64
4/28	FF	331	102	65	92	36	42	331	159	119
4/28	EO	177	109	61	58	27	27	199	143	93

NOTES: FF - First Flush
EO - Extended Overflow
all values in mg/l

TABLE I-9

SUMMARY DISINFECTION DATA - ALL SPRING STORMS

Date	Type	E. Coli per ml	O_3 Dosage mg/1	E. Coli in Effluent per ml	Cl Dosage mg/1	E. Coli in Effluent per ml
4/3	FF	1388	80	21	--	--
4/3	EO	421	59	4	--	--
4/17	EO	1280	40	74	10	0.5
4/20	EO	1850	~30	17	10	1
4/23	EO	6000	<10	3200	10	2
4/28	FF	32000	<10	9500	10	20
4/28	EO	26000	<10	13700	10	8

NOTES: FF - First Flush
EO - Extended Overflow

SUMMARY AND CONCLUSIONS

Table I-10 presents a summary of the various oxidants and oxidation systems utilized. Hydrogen peroxide, chlorine, and ozone were evaluated both alone and in combination. Various catalysts were also utilized including ferrous iron, nickle and cobalt. The best system was a mixture of all three oxidants, which provided up to 70% oxidation of the dissolved fraction of a combined sewer overflow. Any single oxidant did not produce greater than 50% oxidation of the dissolved fraction. When screened overflow was utilized, the highest oxidation percentage was 40%, hence particulate matter was oxidized only to a very limited extent. This indicates that an efficient solids/liquid separation system must be utilized prior to chemical oxidation. Even if this is done a removal efficiency of only about 40% in the chemical oxidation stage could be economically realized. Therefore chemical oxidation of combined sewer overflows cannot be justified.

The following conclusions can be made based on the chemical oxidation studies performed.

1. Chemical oxidation of combined sewer overflow is not technically feasible.
2. Ozone was the best oxidant evaluated in this study.
3. Oxidation with chlorine requires extremely high concentrations of chlorine and results in a high chlorine residual in the effluent.
4. Particulate matter is extremely difficult to oxidize.
5. A combination of oxidants can provide increased oxidation of organic material.

TABLE I-10

SUMMARY OF OXIDATION OF COMBINED OVERFLOWS WITH VARIOUS OXIDANTS AND COMBINATIONS

Line No.	Oxidant	Required Concentration mg/mg(1)	Contact Times (min)	Catalyst Used	Residual Oxidant Remaining mg/1	Type of Organics	COD Removal %	Other Limiting Criteria
1	H_2O_2	3-5	15	Fe^{++}	small	Dissolved	30-50	Requires pH in 3-4 range and removal of iron floc formed during reaction.
2	H_2O_2	5-10	15	Fe^{++}	Large	w/solids	14	
3	Cl	17-20	15-60	None	180	Dissolved	10-15	
4	Cl	30	15-60	None	500	Dissolved	20	Requires extremely high chlorine dosages and hence a dechlorinations step would be required
5	Cl	17	15	UV light	400	Dissolved	40	
6	Cl	17	15-30	UV light	300	w/solids	25-30	
7	O_3	1.2(2)	5	None	Trace	Dissolved	40	
8	O_3	1.2(2)	5	None	None	w/solids	30-40	
9	H_2O_2 Cl & O_3	1.0 1.0 0.6	5	None	Small	Dissolved w/solids	50-70 40	O_3 equipment requires extremely high capital equipment costs at these dosages
10	O_3	1.1	8	3 mg/1 Ni	small	Dissolved	40	
11	H_2O_2 & Cl	30(3)	60	None	Large	Dissolved	7-10	

(1) Milligrams oxidant per mg COD oxidized to obtain stated efficiencies
(2) O_3 dosages are approximate
(3) Equal amounts of Cl and H_2O_2

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APPENDIX II

ANALYTICAL PROCEDURES

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Analytical Instruments and Apparatus

pH Meter: Beckman Model H-2
Beckman Instruments Incorporated
Fullerton, California

Incubator: Model 1483
Precision Scientific Company
Chicago, Illinois

Conductivity Bridge: Model RC16B2
Industrial Instruments Incorporated
Cedar Grove, New Jersey

BOD Incubator: Labline No. 3554B Incubator
Lab-Line Instruments, Incorporated
Melrose Park, Illinois

Analytical Balance: Type H5
Mettler Instruments Corporation
Hightstorm, New Jersey

Ozone Cylinder: Vaporizing Coil and Regulator
Matheson Company, Inc.
East Rutherford, New Jersey

Spectrophotometer: Coleman Model 14
Coleman Instrument Company
Maywood, Illinois

Analytical Procedures and Analyses

The following analyses were performed according to "Standard Methods for the Examination of Water and Wastewater," (35) The page numbers of the analytical procedures used and the modifications to the listed procedures are noted:

Total Solids - Method A, p. 423. The amount of sample used was determined by weight rather than by volume; approximately 20 grams of sample was used for total solids.

Suspended Solids - Method C, p. 424. Reeve-Angel 934AH glass fiber discs were used in place of the mats prepared by using asbestos fibers.

Chemical Oxygen Demand (COD) - p. 510. The standard procedure was used for the analysis of the raw combined overflow. The alternate procedure for dilute samples (p. 513) was used to analyze the raw soluble and oxidized samples.

Biochemical Oxygen (BOD) - p. 415. The BOD tests were performed within several hours after collection. Samples from laboratory oxidation tests were seeded. The raw storm water was used as seed, with a volume of 1 ml of seed. The azide modification of the iodometric method (Method A, p. 406) was used for the analysis of the dissolved oxygen content.

Chloride - Method A, p. 86. A 0.423N standard silver nitrate titrant was used instead of 0.0141N because of high chloride content in raw storm H_2O, thus giving a more definite endpoint.

Chlorine, Total Available - Method A, p. 91.

Ozone Concentration - Method A, p. 220. The titration procedure stated was followed, but the sample collection and ozone absorption techniques are those as specified by the Matheson Company (Matheson Gas Data Book).

Nitrogen (Total Kjeldahl) - p. 44.

Phosphates (ortho) - Method A, page 231. Phosphate analyses were performed after removal of suspended solids by filtration.

pH - p. 226. A Beckman Model H-2 meter was used.

The following analyses were performed according to the procedures listed:

Hydrogen peroxide - This was determined by the iodometric procedure described in Table 3-31 in Handbook of Analytical Chemistry, Edited by Louis Meites, McGraw-Hill Book Company, New York, 1963, p. 3-69.

Coliform Bacteria Count - The millipore filter technique described in "Techniques for Microbiological Analysis, (ADM-40)", (Millipore Filter Corporation, Bedford, Mass., p. 22) was used. Where necessary samples were dechlorinated using Na_2SO_3. (35)

Conductivity - Measurements were made at 25°C using a conductivity cell having a cell constant of 2 cm^{-1}. Measurements were made with a Model RC16B2 Conductivity Bridge made by Industrial Instruments, Inc. according to the procedure described by the manufacturer.

Analysis for Dissolved Matter

The analysis for dissolved matter(i.e.COD, BOD, TOC) was obtained by filtering the sample through a millipore filter (0.47) and then performing the appropriate analysis (COD, BOD, TOC) on the filtrate. If the sample contained gross amounts of solids which would rapidly blind the millipore filter disc, the sample was prefiltered through SS-597 filter paper. The filtrate from this prefiltration was then filtered through the millipore disc as described above.

Dissolved-Air Flotation Test Procedure

A copy of the instructions for performing dissolved-air flotation tests is attached. This sheet described the test as normally used in the REX Chainbelt Process Laboratory. This procedure was modified at times during the chemical oxidation tests by using different detention times and by using ozone instead of normal air as the source of bubbles.

The rate of separation of the suspended solids from a waste is useful in the design of industrial waste treatment equipment. Rate of separation data may be conveniently obtained in the laboratory from treatment tests performed on the waste in question. The treatment processes which will be considered are dissolved-air flotation and sedimentation. Generally, the procedure used in obtaining rate-of-separation data is to observe the solids-liquid interface and to record its travel with time.

A. Dissolved-Air Flotation

In the tests using dissolved-air flotation, the rate of rise of the major portion of the solids is recorded. At times the solids-liquid interface may be vague and good judgment may have to be exercised in following this interface. Care should be taken to avoid following the interface formed by the air bubbles alone. In general, this interface lags behind the solids-liquid interface.

A suggested procedure for the performance of laboratory flotation tests and the equipment needed is as follows:

1. Equipment
    - a. Flotation pressure cell
    - b. Graduated cylinder of one liter capacity containing an effluent sampling arm
    - c. Tire pump or source of compressed air
    - d. Gooch crucibles for suspended solids determinations
    - e. Stop watch

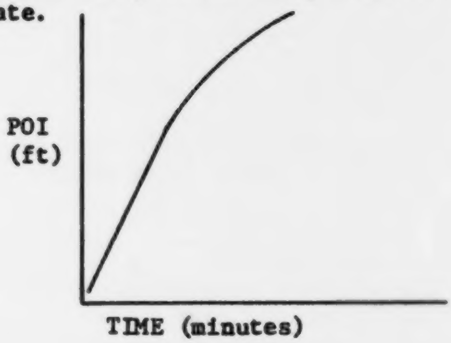
2. Flotation Test Procedure
    - a. Record waste temperature, pH, operating pressure, recycle rate, and flotation detention time
    - b. Record rate of separation data. The form shown below is suggested in obtaining the rate of separation data.

The ultimate data desired is the position of the interface at various intervals throughout the test. The column below labeled "Volume" is used as a convenient means of obtaining the position of the interface at any given time. For example, in the hypothetical case shown below, a liter graduate was used in the test. At the beginning of the test, the solids-liquid interface is at the bottom of the graduate or at zero volume.

As flotation progresses, the solids-liquid interface moves progressively up the height of the graduate. The position of the interface at any given time may be conveniently obtained using the appropriate graduation mark on the liter cylinder as a reference. After the flotation test, the graduation marks may be converted to the feed of height by actual measurement.

Time (min)	Volume (ml)	POI (Position of Interface) (feet)
0	0	0
1	100	0.115
2	350	0.411
3	500	0.589
4	650	0.766
5	800	0.946
6	950	1.122
7	950	1.122
8	950	1.122

The data obtained are plotted using Time as the abscissa and POI in feet as the ordinate.



The slope of the straight line portion of the curve represents the rate of particle rise.

During flotation it should be noted whether settling of solids took place. Note observation.

c. Record the floated scum volume obtained immediately before obtaining a sample of the effluent.

d. Obtain sample of effluent five minutes after flotation is started for the appropriate analyses. Repeat the flotation and obtain another sample of effluent for analysis after an eight minute detention period.

e. If possible, a small portion of the floated scum should be analyzed for total solids content.

APPENDIX III

DEMONSTRATION SYSTEM COSTS

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The following is a breakdown of the equipment and construction costs incurred for completion of the demonstration facility.

RAW FLOW SYSTEM

Primer System	$1092
Pipe Fittings	125
Flanges	178
Elbows	250
Check Valve	194
Ballcentric Valve	236
Raw Feed Pump	3482
	$5557

PRESSURIZED FLOW SYSTEM

Pressurized Flow Pump	$1080
Air Compressor	550
Pipe Fittings	
Elbows	70
Flanges	132
Check Valve	93
8" Ballcentric Valve	170
Pressure Tank	945
Grinnell Pressure Release Valve	295
Air Control System, Complete	950
	$4285

BACKWASH SYSTEM

Nozzles	$ 22
Valve	50
Backwash Pump	270
Elbows	30
	$ 372

DRUM SCREEN ACCESSORIES

Backing Plates	$ 400
Screen Mesh	225
Drive Unit	829
Chain, Sprockets	120
	$1574

SKIMMER ASSEMBLY

Drive Unit	$1300
Chains & Flights, Sprockets	1400
	$2700

CHEMICAL SYSTEM

Pump	$ 886
Tanks	530
Mixer	268
Hoses & Fittings	200
	$1884

ELECTRICAL SYSTEM

Float Switches	$ 140
Panel, Complete Prewired	1710
	$1850

FLOW METERS

Raw Feed Venturi	$ 465
Raw Flow Meter (Recorder)	653
Pressurized Flow Venturi	200
Pressurized Flow Meter	335
Backwash Venturi	103
Backwash Meter (Recorder)	646
Scum Recorder	1060
	$3462

PIPE FOR EFFLUENT AND SCUM

6" Flanges	$ 115
6" Elbows	60
12" Flanges	125
12" Elbows	45
	$ 345

MISCELLANEOUS

Tank Fabrication	$13,418
Tank Erection	20,225
Tank Steel	4,830
	$38,473

MISCELLANEOUS (Continued)

Electrical Power	$ 930
Electrical Wiring Labor & Material	2,307
	$ 3,237
Piping Labor	$ 5,862
Manhole	2,015
	$ 7,877
Foundation	$ 4,150
Building	$ 696
Catwalks & Steps	$ 1,400
GRAND TOTAL	$77,862
Engineering	$12,100

APPENDIX IV

OPERATING DATA
AND STATISTICAL PROCEDURES

TABLE IV-1

OPERATIONAL DATA - 1969

Run No.	Date	Duration (min)	Raw Waste (gal)	Screen Wash (gal)	Floated Scum (gal)	Overflow Rate gpm/sq ft
691	6/4/69	123	378000	0	6200	2.63
692	6/4/69	70	189000	0	3450	2.31
693	6/7/69	103	302400	1600	3400	2.51
694	6/7/69	30	88200	800	900	2.51
695	6/22/69	75	201600	2000	1900	2.30
696	6/25/69	110	340200	<600	3500	2.64
697	6/26/69	150	441000	600	6250	2.51
698	6/27/69	105	277200	600	6700	2.26
699	6/29/69	50	151200	<600	850	2.58
6910	7/2/69	180	554400	<600	4400	2.63
6911	7/11/69	75	226800	800	2100	2.58
6912	7/16/69	35	100800	3200	400	2.46
6913	7/17/69	60	163800	1200	700	2.33
6914	7/23/69	75	226800	5200	1200	2.58
6915	7/26/69	40	100800	<600	450	2.15
6916	8/7/69	50	138600	4800	200	2.37
6917	8/9/69	150	395640	3600	1700	2.25
6918	8/11/69	65	190680	<600	460	2.51
6919	9/4/69	35	100800	1200	450	2.46
6920	9/5/69	90	277200	600	1850	2.63
6921	9/6/69	60	176400	<600	540	2.51
6922	9/23/69	65	201600	600	700	2.65
6923	9/29/69	180	453600	600	2200	2.15
6924	9/29/69	47	143640	<600	310	2.61
6925	10/10/69	65	176400	<600	3850	2.32
6926	10/10/69	243	705600	8400	1700	2.48
6927	10/12/69	75	226800	1200	750	2.58
6928	10/31/69	55	163800	2400	600	2.55
6929	11/2/69	45	113400	1800	250	1.76
6930	11/3/69	110	327600	1800	1150	2.55

TABLE IV-1 (Continued)

OPERATIONAL DATA - 1969

Run No.	Inches	Inches Hour	Operating Pressure scfm	Pressurized Flow Rate gpm	Chemical Addition	
					C-31 mg/1	Cl mg/1
691	0.42	0.13	50	600	0	0
692	0.17	0.68	40	650	0	0
693	0.20	0.15	50	850	0	0
694	0.10	0.10	50	850	0	0
695	0.25	0.32	50	900	0	10
696	0.50	0.50	50	900	0	0
697	1.75	1.17	50	900	0	10
698	0.50	0.40	50	600	0	8
699	1.00	0.70	60	700	0	10
6910	1.50	2.30	60	700	0	10
6911	0.27	0.40	60	475	0	10
6912	0.12	1.50	55	600	0	10
6913	0.45	0.45	50	900	0	10
6914	0.17	0.35	60	400	0	0
6915	0.40	0.35	60	400	0	0
6916	0.10	0.40	60	450	5	0
6917	0.30	0.10	60	600	0	0
6918	0.45	1.60	60	450	3	0
6919	0.10	1.20	60	800	3	0
6920	0.05	2.10	60	650	3	0
6921	0.20	1.20	50	500	3	0
6922	0.14	0.30	50	500	3	0
6923			50	500	5	0
6924	0.70		50	500	0	0
6925	0.17	0.15	50	800	0	0
6926			60	400	3	0
6927	0.40	0.15	50	550	0	0
6928	0.12	0.10	50	550	2.5	0
6929	0.10		50	550	2.5	0
6930	0.50	0.30	50	550	5	0

TABLE IV-1 (Continued)

OPERATIONAL DATA - 1970

Run No.	Date	Duration Min.	Raw Waste Gal.	Screen Wash Gal.	Floated Scum Gal.	Overflow Rate gpm/sq ft High	Overflow Rate gpm/sq ft Low
701	4/13/70	90.7	252000	1200	1050	4.75	3.56
703	4/9/60	47.6	126000	4800	500	4.52	3.39
704	5/11/70	49.7	119700	360	300	4.16	3.09
705	5/12/70	138.6	286020	60	500	3.53	2.65
706	5/15/70	52.3	117180	0	250	3.83	2.87
707	5/22/70	73.7	153720	1380	350	3.56	2.67
708	5/31/70	90.1	186480	3960	1000	3.54	2.65
709	6/1/70	90	178920	420	2050	3.40	2.55
7010	6/12/70	44.1	95760	120	1300	3.71	2.78
7011	6/26/70	75.1	142380	3300	1900	3.24	2.43
7012	6/26/70	68	143640	460	1700	3.61	2.71
7013	7/8/70	49.5	131040	2820	1000	4.52	3.39
7014	7/14/70	40	133560	1080	1000	5.71	4.28
7015	7/15/70	49.8	118440	480	500	4.06	3.05
7016	7/19/70	97.2	217980	60	1600	3.83	2.88
7017	7/27/70	66.6	147420	0	1050	3.79	2.84
7018	7/31/70	34.1	75600	240	150	3.79	2.84
7019	8/18/70	36.4	10200	1740	100	4.79	3.59
7020	9/2/70	51.8	152700	0	600	5.08	3.81
7021	9/3/70	95.6	229320	0	1150	4.10	3.08
7022	10/23/70	38	118440	2160	200	5.33	4.00
7023	10/26/70	64.5	153720	0	500	4.07	3.06
7024	10/27/70	149.1	322560	60	750	3.75	2.81
7025	10/31/70	95	241920	4260	600	4.35	3.27

TABLE IV-1 (Continued)

OPERATIONAL DATA - 1970

Run No.	Rain		Operating Pressure	Pressurized Flow Rate	Chemical Addition (mg/l)		
	inches	in/hr	psig	gpm	C-31	$FeCl_3$	Cl
701	0.7	--	50	800	0	0	10
703	0.3	0.17	50	600	0	0	10
704	0.07	0.09	50	900	6	0	10
705	0.57	0.23	50	900	6	0	10
706	0.25	0.33	50	900	6	0	10
707	0.28	0.36	50	450	6	0	10
708	0.14	0.84	50	450	6	0	10
709	0.5	1.11	50	550	4.2	30	0
7010	0.33	0.11	50	550	4.6	29	0
7011	0.25	0.17	50	450	5.3	17	0
7012	0.10	0.09	60	350	3.9	16	0
7013	0.20	0.80	50	600	4.6	16	0
7014	0.22	0.18	55	950	3.5	21	0
7015	0.10	0.13	50	450	4.5	21	0
7016	0.25	0.13	50	450	4.5	21	0
7017	0.30	0.36	50	450	4.5	21	0
7018	0.45	2.25	50	450	4.5	21	0
7019	0.23	0.15	50	550	3.8	17.5	0
7020	0.25	0.24	50	550	3.8	17.5	0
7021	0.35	0.18	50	550	3.8	17.5	0
7022	0.15	0.10	50	600	0.5*	15	0
7023	0.70	0.35	50	600	0.5*	15	0
7024	0.40	0.10	50	600	0.5*	25	0
7025	0.32	--	50	600	4	25	0

*Herco Floc 810 used for these runs.

TABLE IV-2

RAW WASTE CHARACTERISTICS - FIRST FLUSHES

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	Total Nitrogen	pH	Total Solids	Total Volatile Solids	Ortho Phosphate as PO_4	Coliform per ml
691	676	172	642	391	21.3	7.20	815	431	3.92	62,000
695	615	180	538	310	18.0	7.08	798	480	2.47	110,000
6911	920	180	1180	678	--	6.80	1322	780	--	41,000
6912	472	--	496	253	--	7.2	779	300	4.24	65,000
6914	574	180	554	322	20.7	6.7	833	488	2.88	16,000
6916	531	89	405	310	18.6	6.9	677	443	2.24	150,000
6919	642	170	312	244	23.3	7.0	869	536	5.96	310,000
702	617	224	582	228	18.9	6.90	913	494	2.00	600,000
708	512	160	431	247	13.3	7.1	746	387	2.15	110,000
7011	390	212	232	156	8.8	7.1	732	433	0.68	16,650
7013	649	330	479	289	13.9	7.0	984	604	1.06	4,400
7022	372	145	415	273	19.5	7.35	--	--	2.46	220,000

NOTE: All values in mg/l except pH.

TABLE IV-3

RAW WASTE CHARACTERISTICS - EXTENDED OVERFLOWS - 1969 DATA

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	Total Nitrogen	pH	Total Solids	Total Volatile Solids	Ortho Phosphate	Coliform per ml
691	108	38	112	58	4.1	7.10	244	121	1.18	9,500
692	220	43	365	145	8.1	6.80	502	215	1.76	39,000
693	241	--	169	117	--	7.60	560	302	--	--
694	173	--	158	80	--	7.60	312	168	--	--
695	207	65	168	102	6.4	7.10	302	176	0.88	36,000
696	87	17	88	46	2.3	6.80	188	102	0.60	5,700
697	66	7	192	55	2.0	7.20	157	90	0.30	1,300
698	47	7	57	29	1.4	7.45	170	86	0.37	7,800
699	59	--	148	43	--	7.00	208	70	--	6,200
6910	78	12	208	66	3.1	7.10	280	102	0.36	19,000
6911	188	49	276	105	--	6.40	384	162	--	20,000
6913	223	--	176	120	--	7.45	541	250	0.52	38,000
6915	118	--	116	62	--	7.30	197	118	0.24	1,300
6917	185	--	104	78	7.6	7.10	445	222	3.26	--
6918	137	37	192	82	4.8	7.00	278	170	0.50	1,500
6920	127	31	114	62	4.3	6.85	247	160	0.30	160,000
6921	114	25	102	64	4.6	6.80	226	108	0.78	55,000
6922	248	--	165	117	8.1	7.20	478	296	1.00	82,000
6923	163	61	104	68	8.4	7.18	279	152	1.20	1,400
6924	125	31	129	66	3.8	7.18	218	105	0.91	330
6925	118	39	70	22	8.8	7.60	478	225	1.24	55,000
6926	95	29	141	65	3.3	7.10	199	82	0.36	12,000
6927	94	36	56	45	--	7.20	269	128	0.72	78,000
6928	334	124	180	138	10.2	6.90	450	272	2.12	200,000
6929	195	70	87	63	9.1	7.30	457	244	1.88	180,000
6930	217	65	87	66	7.0	7.40	390	224	2.26	110,000

NOTE: All values in mg/l except pH.

TABLE IV-3 (Continued)

RAW WASTE CHARACTERISTICS - EXTENDED OVERFLOWS - 1970 DATA

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	Total Nitrogen	pH	Total Solids	Total Volatile Solids	Ortho Phosphate as PO_4	Coliform per ml
701	143	36	151	75	2.9	7.45	295	112	0.83	--
703	206	91	218	88	7.4	7.1	412	200	1.34	270,000
704	141	58	109	39	5.4	7.4	676	267	1.08	12,200
705	199	35	316	193	6.3	6.8	511	203	1.1	--
706	140	28	149	75	3.7	7.7	466	246	0.5	700
707	211	101	228	146	7.5	6.9	533	298	1.17	340,000
709	168	37	148	86	4.4	7.35	343	160	0.92	73,000
7012	177	67	119	81	6.3	7.2	577	291	0.96	12,000
7014	191	--	232	139	5.55	7.2	441	220	0.06	35,500
7015	162	--	140	85	4.9	6.9	415	177	0.47	118,800
7016	130	--	135	95	--	7.1	304	145	0.71	7,660
7018	244	--	337	183	--	7.1	566	246	--	--
7019	286	113	264	164	6.95	7.4	574	250	2.28	3,800
7020	153	53	466	204	3.4	7.15	673	304	0.74	160,000
7021	90	25	118	63	2.4	7.05	266	116	0.77	4,800
7023	169	67	107	73	8.7	7.7	--	--	2.73	55,000
7024	178	54	171	125	4.9	7.1	--	--	0.74	26,000
7025	139	66	123	103	6.4	7.4	--	--	1.78	117,000

NOTE: All values in mg/l except pH.

TABLE IV-4

SCREENED WATER QUALITY - FIRST FLUSHES

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	pH	Total Solids	Total Volatile Solids
695	168	51	170	91	7.75	833	372
6912	449	--	424	234	7.0	725	300
6914	281	120	260	150	7.0	536	302
6916	432	71	290	290	6.9	290	369
6919	510	169	286	236	6.8	848	520
702	321	133	306	126	7.30	716	303
708	306	93	272	128	7.3	623	293
7011	254	141	180	106	6.8	665	394
7013	611	295	418	231	6.8	906	552
7022	262	113	331	173	7.4	--	--

NOTE: All values in mg/l except pH.

TABLE IV-5

SCREENED WATER QUALITY - EXTENDED OVERFLOW - 1969 DATA

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	pH	Total Solids	Total Volatile Solids
693	132	--	123	69	7.70	463	234
694	165	--	124	64	7.40	290	156
695	159	50	148	77	6.92	301	165
696	49	15	68	36	7.00	184	94
697	26	7	152	42	7.20	226	84
698	51	5	40	24	7.40	142	79
699	49	--	132	32	6.90	196	77
6910	60	9	156	50	7.25	258	100
6911	144	37	249	107	6.80	330	173
6913	163	--	111	38	6.90	231	100
6915	89	--	67	39	7.10	133	70
6917	141	--	62	47	7.12	404	224
6918	103	31	139	55	7.05	244	147
6920	76	21	45	32	6.93	205	150
6921	91	--	88	51	6.90	231	105
6922	184	--	99	74	7.25	396	242
6923	138	54	95	60	7.22	273	125
6924	105	27	124	57	7.10	189	85
6925	122	41	79	26	7.62	422	194
6926	73	20	96	45	7.10	169	76
6927	78	28	47	35	7.30	289	129
6928	260	90	105	55	6.90	412	236
6929	171	59	72	54	7.40	411	226
6930	200	60	71	44	7.30	369	202

NOTE: All values in mg/l except pH.

TABLE IV-5 (Continued)

SCREENED WATER QUALITY - EXTENDED OVERFLOW - 1970 DATA

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	pH	Total Solids	Total Volatile Solids
701	133	35	138	66	7.5	294	106
703	183	75	180	95	7.2	372	193
704	126	15	52	16	7.6	635	270
705	121	26	198	122	7.2	469	174
706	90	18	86	44	7.6	410	213
707	156	51	126	61	7.2	487	207
709	84	18	55	39	6.6	344	183
7012	123	49	104	53	7.1	404	189
7014	183	--	226	114	6.8	441	213
7015	137	--	132	63	6.7	439	205
7016	80	--	98	56	6.6	281	141
7018	169	--	272	125	6.8	510	214
7019	201	71	202	108	7.4	543	303
7020	122	35	252	86	7.1	528	192
7021	57	13	131	156	6.9	298	148
7023	136	58	99	73	7.6	--	--
7024	82	29	85	64	--	--	--
7025	132	47	161	101	6.6	--	--

NOTE: All values in mg/l except pH.

TABLE IV-6

EFFLUENT WATER QUALITY - FIRST FLUSHES

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	Total Nitrogen	pH	Total Solids	Total Volatile Solids	Ortho Phosphate as PO_4	Coliform per ml
691	177	74	166	92	15.4	7.50	714	320	8.10	26,000
6911	137	39	130	62	---	7.00	411	221	---	40
6912	245	--	224	102	---	8.00	902	437	2.51	---
6914	179	64	109	57	9.0	7.10	533	289	1.84	16,000
6916	173	51	89	64	9.0	7.12	627	314	1.10	80,000
6919	263	94	108	96	13.5	7.53	959	460	3.02	8
702	64	24	102	61	2.7	7.7	1251	481	0.46	1
702(1)	63	21	84	51	2.4	7.6	1183	410	0.22	1
708	168	66	105	57	7.4	7.2	440	206	1.47	180
708(1)	197	103	145	73	8.2	7.2	519	248	2.79	1500
7011	143	85	70	38	4.0	6.8	545	297	0.08	5700
7011(1)	157	76	60	33	4.0	6.8	412	253	0.03	5700
7013	285	149	135	58	7.5	7.0	690	406	0.07	3150
7013(1)	337	178	165	90	8.1	7.1	676	389	0.06	500
7022	73	18	65	47	5.4	7.6	---	---	1.32	---
7022(1)	68	22	59	48	4.6	7.6	---	---	1.01	1400

(1) Overflow rate ∿ 3.75 gpm/sq ft

All values in mg/l except pH & coliform
Overflow Rate ∿ 2.5 gpm/sq ft except where noted

TABLE IV-7

EFFLUENT WATER QUALITY - EXTENDED OVERFLOW - 1969 DATA

Run #	COD	BOD	Suspended Solids	Volatile Suspended Solids	Total Nitrogen	pH	Total Solids	Total Volatile Solids	Ortho Phosphate	Coliform per ml
691	107	36	120	58	5.8	7.25	320	147	2.15	9,000
692	122	25	218	76	6.2	6.60	380	172	1.76	61,000
693	60	--	59	14	--	8.15	725	306	---	---
695	105	52	95	43	4.2	7.60	606	296	1.13	<1
696	50	13	58	24	2.2	7.10	258	140	0.90	<0.04
697	25	7	138	38	1.3	7.20	234	84	0.40	<0.04
698	30	11	63	30	1.2	7.45	174	95	0.40	<0.04
699	34	--	89	11	--	7.10	194	77	--	0.02
6910	51	6	142	40	1.8	7.25	300	125	0.44	<0.02
6911	102	25	194	59		7.10	286	206		0.1
6913	134		65	44		7.70	412	220	1.77	<50
6915	69		55	34		7.90	434	203	0.80	10,000
6917	122		38	31	10.5	7.20	429	240	2.82	---
6918	67	22	34	15	4.5	7.10	475	185	3.86	1,000
6920	57	15	34	27	4.3	6.98	134	73	0.58	400
6921	63	6	44	32	4.3	7.00	222	98	0.65	15
6922	131		56	46	5.6	7.50	544	282	1.2[illegible]	150
6923	99	43	49	37	7.2	7.32	250	109	0.81	8
6924	72	24	67	31	4.0	7.20	156	56	0.83	30
6925	54	24	38	10	6.2	7.85	457	214	1.22	19,000
6926	30	8	35	17	2.3	7.00	115	46	0.42	8,200
6927	45	17	22	17	--	7.10	221	100	0.61	55,000
6928	189	73	91	53	6.8	7.00	335	184	1.58	120,000
6929	149	48	48	34	6.9	7.30	358	176	1.42	69,000
6930	207	54	54	38	6.9	7.30	342	182	1.96	110,000

All values in mg/l except pH and coliform

TABLE IV-7 CONTINUED

EFFLUENT WATER QUALITY - EXTENDED OVERFLOW - 1970 DATA HIGH OVERFLOW RATES

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	Total Nitrogen	pH	Total Solids	Total Volatile Solids	Ortho Phosphate	Coliform per ml
701	86	24	110	49	2.9	7.7	519	168	.58	5,000
703	203	74	161	89	6.4	7.6	541	236	1.9	<1
704	102	26	73	37	5.4	7.6	702	290	1.5	<1
705	51	11	65	38	4.5	7.6	341	127	1.4	3
706	39	13	50	27	3.0	7.6	390	194	0.6	.06
707	121	42	114	56	5.7	7.3	385	189	1.38	33
709	66	18	35	23	3.4	6.8	279	139	0.09	>10,000
7012	113	46	76	36	4.8	6.8	465	234	0.01	11,200
7014	135	---	78	45	5.4	6.9	421	199	0.03	8,200
7015	99	---	84	44	3.6	6.6	349	168	0.03	49,500
7016	38	---	23	17	--	6.6	184	91	0	1,960
7018	67	---	47	39	--	6.8	295	137	---	---
7019	71	23	67	35	2.5	8	751	305	0.28	600
7020	56	29	172	63	2	7.4	683	297	0.06	73,000
7021	30	5	39	20	1.8	7.0	184	108	0.12	1,700
7023	92	36	70	47	6.1	7.6	---	---	0.31	49,000
7024	87	29	75	38	3.6	7.3	---	---	0.43	15,300
7025	57	24	39	31	4	6.6	---	---	0.06	6,000

All values in mg/l except pH and coliform

TABLE IV-7 CONTINUED

EFFLUENT WATER QUALITY - EXTENDED OVERFLOW 1970 DATA - LOW OVER RATES

Run No.	COD	BOD	Suspended Solids	Volatile Suspended Solids	Total Nitrogen	pH	Total Solids	Total Volatile Solids	Ortho Phosphate	Coliform per ml
701	98	23	94	40	2.9	7.8	535	174	.37	5,800
703	184	72	141	79	2.7	7.6	508	229	2.0	<1
704	92	18	25	4	5.3	7.7	616	241	1.1	<1
705	55	11	65	40	4.5	7.6	311	123	---	9
706	33	7	29	20	2.9	7.6	445	237	0.4	.02
707	102	39	73	41	5.2	7.3	343	170	1.29	38
709	57	18	26	21	3.4	6.8	274	152	0.06	>10,000
7012	104	43	57	34	4.4	6.8	499	243	0.05	14,200
7014	122	---	74	44	3.9	7.0	373	183	0.03	6,330
7015	92	---	88	41	2.6	6.7	350	163	0.03	28,000
7016	34	---	20	15	--	6.7	200	91	0	1,400
7018	61	---	46	30	---	6.8	285	116	---	---
7019	102	34	76	37	3.1	7.7	621	274	0.95	2,100
7020	49	12	71	29	1.9	7.2	567	248	0.18	32,000
7021	38	6	33	19	1.8	7.0	198	116	0.09	1,400
7023	107	33	55	37	5.3	7.4	---	---	0.15	38,000
7024	69	26	59	49	3.8	7.1	---	---	0.43	14,400
7025	69	25	51	40	3.9	6.6	---	---	0.06	28,000

All values in mg/1 except pH and coliform

TABLE IV-8

DISSOLVED COD AND TOC DATA

Run No.	Raw Waste Dissolved COD	Raw Waste Total TOC	Raw Waste Dissolved TOC	Screened Effluent Dissolved COD	Screened Effluent Total TOC	Screened Effluent Dissolved TOC	System Effluent Dissolved TOC	System Effluent Total TOC	System Effluent Dissolved TOC
701	32	35	12	33	31	12	27	24	8
702	147	237	57	107	121	41	64	23	18
703	103	75	39	97	53	37	122	74	48
704	52	42	18	63	38	20	59	31	20
705	47	71	16	54	36	15	48	16	15
706	49	41	17	44	29	14	34	18	12
707	73	--	--	84	--	--	66	--	--
708	102	--	--	117	--	--	105	--	--
709	54	--	--	38	--	--	43	--	--
7011	154	--	--	147	--	--	113	--	--
7012	68	--	--	71	--	--	80	--	--
7013	313	249	128	336	235	137	219	117	86
7014	101	--	--	89	--	--	70	--	--
7015	55	--	--	52	--	--	39	--	--
7016	28	--	--	26	--	--	27	--	--
7018	36	79	--	28	61	--	33	21	--
7019	92	139	36	94	91	31	62	44	24
7020	47	38	13	36	27	12	49	7	9
7021	45	--	17	46	--	17	34	--	13
7022	60	--	--	59	--	--	34	--	--
7023	71	--	--	56	--	--	56	--	--
7024	37	--	--	34	--	--	38	--	--
7025	55	57	25	45	49	18	43	27	14

TABLE IV-9

SCREEN BACK WASH & FLOATED SCUM QUALITY
1969 DATA

Run #	Screen Back Wash Quality pH	Total Solids	Total Volatile Solids	Suspended Solids[1]
693	--	--	--	--
694	--	--	--	--
695	6.8	1688	1388	1554
696	6.8	643	538	543
697	--	--	--	--
698	7.8	296	220	183
699	--	--	--	--
6910	7.1	1443	1025	1371
6911	6.5	2542	1814	2434
6912	7.0	2192	1450	1909
6913	7.2	1865	1100	1500
6914	7.0	--	--	--
6915	--	--	--	--
6916	6.9	3638	2486	3366
6917	6.6	1776	1406	1435
6918	--	--	--	--
6919	6.8	2303	1830	1746
6920	7.1	2431	1813	2298
6921	--	--	--	--
6922	7.1	4185	2533	3872
6923	7.2	890	694	715
6924	--	--	--	--
6925	--	--	--	--
6926	7.4	2901	972	2843
6927	--	--	--	--
6928	6.8	1600	1236	1330
6929	7.2	1623	1244	1253
6930	7.3	1322	1048	1019

1. Calculated based on dissolved solids in raw waste

All values in mg/l except pH

TABLE IV-9 (Continued)

SCREEN BACK WASH & FLOATED SCUM QUALITY
1969 DATA

	Floated Scum Quality			
Run #	pH	Total Solids	Total Volatile Solids	Suspended Solids[1]
693	6.8	2395	1520	2004
694	—	—	—	—
695	7.2	4005	2390	3879
696	7.1	584	330	484
697	7.2	686	332	613
698	7.5	385	202	272
699	—	—	—	—
6910	7.3	1439	647	1367
6911	6.6	36860	19958	36720
6912	7.0	14282	7550	13999
6913	7.0	19081	10700	18716
6914	—	11131	6919	10852
6915	7.0	3705	2244	3624
6916	6.6	22627	13403	22355
6917	6.8	4687	3163	4346
6918	7.0	7694	3544	7608
6919	6.8	13650	8450	13093
6920	7.0	12168	6644	12035
6921	6.9	3111	1408	3087
6922	7.0	2291	1848	1978
6923	6.9	13228	8082	13053
6924	7.1	6699	3471	6610
6925	7.5	2508	1097	2100
6926	7.3	11361	5002	11303
6927	7.0	1705	1132	1492
6928	6.8	3268	2100	2998
6929	6.8	7027	5045	6657
6930	7.0	2804	2020	2501

1. Calculated based on dissolved solids in raw waste

All values in mg/l except pH

TABLE IV-10

SCREEN BACKWASH AND FLOATED SCUM QUALITY 1970 - DATA

Run No.	Screen Backwash				Floated Scum			
	Screen pH	Total Solids	Total Volatile Solids	Suspended Solids (1)	pH	Total Solids	Total Volatile Solids	Suspended Solids (1)
701	7.7	1655	913	1435	7.5	2547	1455	2403
702	7.2	3929	3017	3599	7.3	8044	4549	7713
703	6.9	1891	1310	1697	7.2	4431	2284	4237
704	---	---	---	---	7.4	7374	4408	6807
705	---	---	---	---	7.2	34981	9835	34663
706	---	---	---	---	7.5	8399	4484	8082
707	7.1	3522	2723	3142	6.9	11379	5157	11074
708	---	---	---	---	7.2	21201	9074	20866
709	6.6	3976	3073	3781	6.6	27416	10406	27221
7011	6.7	3459	2387	2959	6.6	11601	5618	11101
7012	---	---	---	---	6.8	15675	7476	15217
7013	6.9	2817	2068	2312	6.8	13427	6829	12922
7014	---	---	---	---	6.9	6014	2606	5805
7015	---	---	---	---	6.6	15452	7005	15177
7016	6.7	1191	893	1022	6.6	13812	5747	13643
7018	7.1	2945	1987	2716	7.0	17062	6249	16836
7019	7.1	2307	1636	1940	7.5	3138	1589	2771
7020	---	---	---	---	---	---	---	---
7021	---	---	---	---	---	16300	4360	16152
7022	---	---	---	---	7.2	24064	9781	---
7023	---	---	---	---	7.4	12709	6024	---
7024	---	---	---	---	7.1	3782	1807	---
7025	6.7	---	---	---	6.6	24026	11683	---

(1) Calculated based on dissolved solids in raw waste
All values in mg/l except pH

TABLE IV-11

FIRST FLUSH EVALUATIONS
(RAW COMBINED SEWER OVERFLOW QUALITY)

Days Since Last Overflow	Run No.	COD mg/1	BOD mg/1	Suspended Solids mg/1	Volatile Suspended Solids mg/1
0	703	206	91	218	88
0	692	220	43	365	145
0	694	173	--	158	80
0	6924	125	31	129	66
0	6930	217	65	87	66
0	705	199	35	316	193
0	709	168	37	148	86
0	7012	117	49	119	81
1	697	66	7	---	26
1	698	47	7	57	29
1	6913	223	---	176	120
1	6920	127	31	114	62
1	6921	114	25	102	64
1	6926	95	29	141	65
1	6927	94	36	56	45
1	7015	191	---	140	85
1	7021	90	25	118	63
1	7024	178	54	171	125
2	699	59	---	148	43
2	6911	185	---	104	78
2	706	140	28	149	75
2	704	141	58	109	39
2	706	140	28	149	75
2	7023	169	67	107	73
3	693	241	---	169	117
3	696	87	17	88	46

TABLE IV-11 CONTINUED

FIRST FLUSH EVALUATIONS
(RAW COMBINED SEWER OVERFLOW QUALITY)

Days Since Last Overflow	Run No.	COD mg/1	BOD mg/1	Suspended Solids mg/1	Volatile Suspended Solids mg/1
3	6915	118	---	116	62
3	6925	195	70	87	63
3	7016	130	---	135	95
3	7018	244	---	337	183
3	7025	139	66	123	103
4	6910	78	12	208	66
5	6912	472	---	496	253
5	7014	191	101	232	139
6	6914	574	18	554	322
6	6923	163	61	104	68
6	707	211	101	228	146
8	6911	920	180	1180	678
8	708	512	160	431	247
11	6918	137	37	192	82
11	6925	118	39	70	22
11	7013	649	330	479	289
12	6916	531	89	405	310
13	7011	390	212	232	156
14	7020	153	53	466	204
15	695	615	180	538	310
17	6922	248	---	165	117
17	7019	286	113	264	164
18	691	676	172	642	391
19	6928	334	124	180	138
19	7022	372	145	415	273
24	6919	642	170	312	244
26	702	617	224	582	228

TABLE IV-12

FIRST FLUSH REMOVALS IN PERCENT

	Screen				Overall System				
Run #	COD	BOD	SS	VSS	COD	BOD	SS	VSS	N
691	--	--	--	--	73.8	57.0	74.1	76.5	27.7
695	72.7	71.7	68.4	70.6	--	--	--	--	--
6911	--	--	--	--	85.1	78.3	89.0	90.9	--
6912	4.9	--	14.5	7.5	48.1	--	54.8	59.7	--
6914	51.1	33.3	53.1	53.4	68.8	64.4	80.3	82.3	56.5
6916	18.6	20.2	28.4	17.1	67.4	42.7	78.0	79.4	51.6
6919	20.6	0.6	8.3	3.3	59.0	44.7	65.4	60.7	42.1
702[1]	--	--	--	--	89.7	90.6	85.6	77.6	87.3
702	48.0	40.6	47.4	44.7	89.6	89.2	82.5	73.3	85.7
708	40.2	41.9	36.9	48.2	67.2	58.8	75.6	76.9	44.4
708[1]	--	--	--	--	61.5	35.6	66.4	70.5	38.3
711	34.9	33.5	22.4	32.1	63.3	59.9	69.8	75.6	55.0
711[1]	--	--	--	--	59.7	64.2	74.1	78.9	55.0
713	5.9	10.6	12.7	20.1	56.1	54.9	71.8	79.9	46.0
713[1]	--	--	--	--	48.1	46.1	65.6	68.9	41.7
722	29.6	22.1	20.2	29.3	80.4	87.6	84.3	82.8	72.3
722[1]	--	--	--	--	81.7	84.8	85.8	82.4	76.4

Overflow rate ∿ 2.5 gpm/sq. ft. except where noted

1. Overflow rate ∿ 3.75 gpm/sq. ft.

TABLE IV-13

EXTENDED OVERFLOW REMOVALS IN PERCENT - 1969 DATA

	Screen				Overall System				
Run #	COD	BOD	SS	VSS	COD	BOD	SS	VSS	N
69-2	---	---	---	---	44.5	41.9	40.3	47.6	23.5
69-3	45.2	---	27.2	41.0	75.1	---	65.1	88.0	---
69-4	4.6	---	21.5	20.0	---	---	---	---	---
69-5	23.2	23.1	11.9	24.5	49.3	20.0	43.5	57.8	34.4
69-6	43.7	11.7	22.7	21.7	42.5	23.5	34.1	47.8	4.3
69-7	60.6	0	20.8	23.6	62.1	0	28.1	30.9	35.0
69-8	---	28.6	29.8	17.2	36.2	---	10.5	3.4	14.3
69-9	16.9	---	10.8	25.6	42.4	---	39.9	74.4	---
69-10	23.1	25.0	25.0	24.2	34.6	50.0	31.7	39.4	41.9
69-11	23.4	24.5	9.8	43.8	45.7	49.0	29.7	43.8	---
69-13	26.9	---	36.9	68.3	39.9	---	63.1	63.3	---
69-15	24.6	---	42.2	37.1	41.5	---	52.6	45.2	---
69-17	23.8	---	40.4	39.7	34.1	---	63.5	60.3	---
69-18	24.8	16.2	27.6	32.9	51.1	40.5	82.3	81.7	6.2
69-20	40.2	32.3	60.5	48.4	48.8	51.6	70.2	56.5	
69-21	20.2	---	13.7	20.3	44.7	76.0	56.9	50.0	6.8
69-22	25.8	---	40.0	36.8	47.2	---	66.1	60.7	30.9
69-23	15.3	11.5	8.6	11.7	39.3	29.5	52.9	45.6	14.3
69-24	16.0	12.9	3.8	13.6	42.4	22.6	48.1	53.0	---
69-25	3.4	5.1	12.9	18.2	54.2	38.5	45.7	54.5	29.5
69-26	23.2	31.0	31.9	30.7	68.4	72.4	75.2	73.8	30.3
69-27	17.0	22.2	16.1	22.2	52.1	52.8	60.7	62.2	---
69-28	22.2	27.4	41.7	60.1	43.4	41.1	49.4	61.6	33.3
69-29	12.3	15.7	17.2	14.3	23.6	31.4	44.8	46.0	24.2
69-30	7.8	7.7	18.4	33.3	4.6	16.9	37.9	42.4	1.4

TABLE IV-13 CONTINUED

EXTENDED OVERFLOW REMOVALS IN PERCENT 1970 DATA - HIGH OVERFLOW RATES

Run	Overall System				
No.	COD	BOD	SS	VSS	N
701	39.9	33.3	27.2	34.7	0
703	1.5	18.7	26.7	1.1	13.5
704	27.7	55.2	33.0	51.2	0
705	74.4	68.6	79.4	80.3	27.7
706	72.1	53.6	66.4	64.0	23.0
707	42.7	58.4	50.0	61.6	24.0
709	60.7	51.4	76.4	73.3	22.7
7012	36.2	31.4	36.1	55.6	23.8
7014	29.3	---	66.4	67.6	3.6
7015	38.9	---	40.0	48.2	26.5
7016	70.8	---	83.0	82.1	---
7018	72.5	---	86.1	78.7	---
7019	75.2	79.7	74.6	78.7	64.0
7020	63.4	45.3	63.1	69.1	41.2
7021	66.7	80.0	67.0	68.3	25.0
7023	45.6	46.3	34.6	35.6	29.9
7024	51.1	46.3	56.1	69.6	26.5
7025	60.0	63.6	68.3	69.9	37.5

TABLE IV-13 CONTINUED

EXTENDED OVERFLOW REMOVALS IN PERCENT 1970 DATA - LOW OVERFLOW RATES

Run	Screen				Overall System				
No.	COD	BOD	SS	VSS	COD	BOD	SS	VSS	N
701	7.0	2.8	8.6	12.0	31.5	36.1	37.7	46.7	0
703	11.2	17.6	17.4	---	10.7	20.9	35.3	10.2	63.5
704	10.6	74.1	52.3	59.0	34.8	69.0	77.1	89.7	0
705	39.2	25.7	37.3	36.8	72.4	68.6	79.4	79.3	27.8
706	35.7	35.7	42.3	41.3	74.6	75.0	80.5	73.3	18.8
707	26.1	49.5	44.7	58.2	51.7	61.4	68.0	71.9	30.7
709	50.0	51.4	62.8	54.7	66.1	51.4	82.4	75.6	22.7
7012	30.5	26.9	12.0	34.6	41.2	35.8	52.1	58.0	30.2
7014	4.2	10.6	2.6	18.0	36.1	---	68.1	68.4	29.5
7015	15.4	---	5.7	25.9	43.2	---	37.1	51.8	46.9
7016	38.0	---	27.0	32.0	73.8	---	85.2	84.2	---
7018	31.0	---	19.0	32.0	75.1	---	86.4	83.6	---
7019	30.0	37.0	23.0	34.0	64.3	69.9	71.2	77.4	55.4
7020	20.0	34.0	46.0	58.0	68.0	77.4	84.8	85.8	44.1
7021	37.0	48.0	---	11.0	57.8	76.0	72.0	69.8	25.0
7023	19.5	13.4	7.5	---	36.7	50.7	48.6	49.3	39.2
7024	53.9	46.3	50.3	48.8	62.1	51.9	65.5	60.8	23.5
7025	5.0	28.8	---	2.0	50.4	62.1	58.5	61.2	39.1

TABLE IV-14

SUSPENDED SOLIDS MASS BALANCE - 1969 DATA

Run No.	Pounds SS In	Pounds SS Out	Pounds SS Left in Tank	Pounds SS Screen Wash	Pounds SS Scum	Balance %(1)
691	353	372	80	0	---	-28
692	575	337	145	0	---	+16
693	426	146	39	--	58	+57
695	282	157	63	26	63	- 9
696	250	163	39	--	14	+14
697	706	499	92	--	31	+12
698	132	141	42	--	15	-50
699	187	111	59	--	--	+ 9
6910	961	650	94	--	50	+17
6911	522	362	86	16	84	- 5
6912	417	181	48	51	47	+22
6913	240	88	43	15	109	- 6
6915	97	46	37	--	15	- 1
6917	343	124	25	43	62	+26
6920	263	79	23	12	191	-16
6921	150	64	29	--	13	+29
6922	277	94	37	19	12	+41
6923	393	184	33	4	239	-17
6924	154	80	45	--	17	+ 8
6925	103	54	25	--	68	-43
6926	829	203	23	199	160	+29
6927	106	41	15	--	11	+37
6928	246	122	60	13	15	+14
6929	82	45	32	19	17	-37
6930	238	146	36	15	25	+ 7

(1) (+) Excess solids in; (-) Excess solids out

TABLE IV-14 CONTINUED

SUSPENDED SOLIDS MASS BALANCE - 1970 DATA

Run No.	Pounds SS In	Pounds SS Out	Pounds SS Left In Tank	Pounds SS Screen Wash	Pounds SS Scum	Balance %(1)
701	316	211	68	14	21	+ 1
703	228	151	100	67	18	-47
704	108	48	33	--	17	+ 9
705	750	154	43	--	144	+55
706	145	38	26	--	17	+44
707	291	120	63	36	32	+14
708	667	181	80	--	173	+35
709	220	44	20	38	463	-157
7011	274	74	43	81	175	-36
7012	142	76	43	--	215	-135
7013	481	158	100	54	107	+13
7014	257	83	50	--	48	+30
7015	138	84	57	--	63	-48
7016	244	40	15	5	181	+ 1
7018	211	29	31	54	21	+36
7019	224	60	48	28	2	+35
7021	225	68	24	--	154	- 9
7022	408	60	41	--	40	+65
7023	137	80	42	--	53	-28
7024	458	179	44	--	24	+46
7025	247	89	30	--	120	+ 3

(1) + Excess solids in
- Excess solids out

STATISTICAL PROCEDURES

The following statistical procedures were utilized:

Estimate of a Mean Value

$$x = \bar{x} \pm \frac{ts}{\sqrt{n}}$$

where: x - estimated mean value

$\bar{x}$ = mean of n values

t = student "t" value at n-1 degrees of freedom for some confidence level

s = standard deviation of the data points

n = number of pieces of data

Comparison of Means Test

1. Perform "F" test

$$F_c = \frac{s_1^2}{s_2^2}$$

If: $F_c < F_t$ Go to Step 2

If: $F_c > F_t$ Go to Step 3

F_t = Table "F" value

F_c = calculated "F" value

s_1 = standard deviation of first set of data

s_2 = standard deviation of second set of data

2. Comparison of means "t" test when:

$$\sigma_1^2 = \sigma_2^2$$

$$s = \sqrt{\frac{\Sigma(x_1)^2 - ((\Sigma x_1)^2/n_1) + \Sigma(x_2)^2 - ((\Sigma x_2)^2/n)}{n_1 + n_2 - 2}}$$

$$t = \frac{\bar{x}_1 - \bar{x}_2}{\sqrt{(1/n_1) + (1/n_2)}}$$

Degrees of freedom (DF) = $n_1 + n_2 - 2$

Read "t" table at t and DF to find confidence level at which a difference exists

3. Comparison of means "t" test when

$$\sigma_1^2 \neq \sigma_2^2$$

$$S\bar{x}_1^2 = (S_1)^2/n_1$$

$$S\bar{x}_2^2 = (S_2)^2/n_2$$

Complete DF (ν) from:

$$1/\nu = (1/\nu_1)((S\bar{x}_1^2)/(S\bar{x}_1^2 + S\bar{x}_2^2))^2 + (1/\nu_2)((S\bar{x}_2^2)/(S\bar{x}_1^2 + Sx_2^2))^2$$

where: $\nu_1 = N_1 - 1$

$\nu_1 = N_2 - 2$

$$t = \frac{\bar{x}_1 - \bar{x}_2}{\sqrt{S\bar{x}_1^2 + S\bar{x}_2^2}}$$

Read "t" table at t and DF to find confidence level at which a difference exists

Paired Comparison Test

1. Calculate S_{di}

where $S_d = \sqrt{\dfrac{\Sigma(dj - \bar{d})^2}{n-1}}$

and $\bar{d} = \bar{x} - \bar{y}$

$d_j = x - y$

n = number of pairs of data

x & y = data pair

$\bar{x}$ & $\bar{y}$ = average of each set of data

2. Calculate t;

where $t = \dfrac{\bar{d}\sqrt{n}}{s_d}$

Compare calculated "t" to table "t" to determine if there is a significant difference in the two sets of data.

Confidence Range

The Confidence Range was calculated by:

$\text{Range} = \bar{x} \pm ts/\sqrt{n-1}$

where $\bar{x}$ = mean value

t = value from "t" table at desired confidence range and degrees of freedom

s = standard deviation

n = number of pieces of data